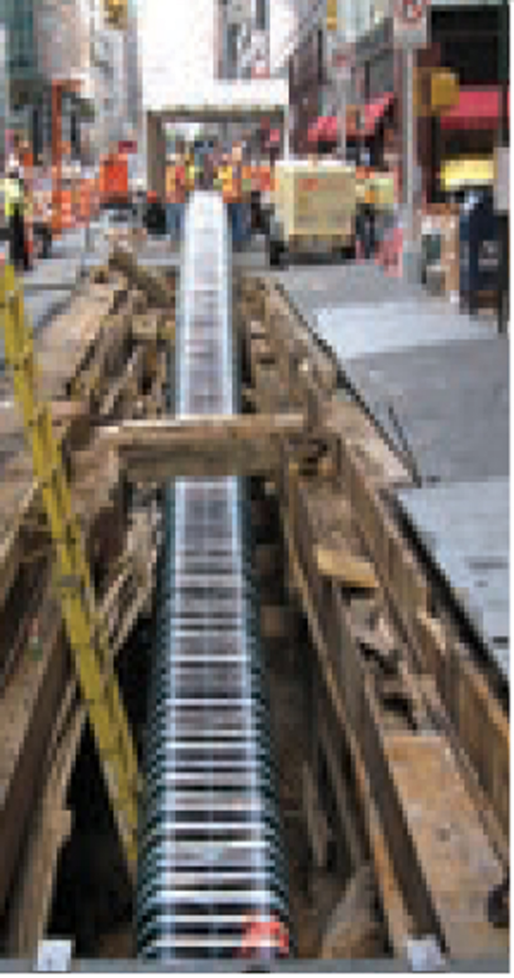

Pipelines 2015

Recent Advances in Underground Pipeline Engineering and Construction

Edited by
V. Firat Sever, Ph.D., P.E.
Lynn Osborn, P.E.

ASCE

PIPELINES 2015

RECENT ADVANCES IN UNDERGROUND PIPELINE ENGINEERING AND CONSTRUCTION

PROCEEDINGS OF THE PIPELINES 2015 CONFERENCE

August 23–26, 2015
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Preface

It is an exciting time to be in the pipeline industry. The importance of pipeline infrastructure can never be over emphasized; from supplying drinking water to collecting wastewater, conveyance of petroleum products and other fluids (or for some cases solids as well), we depend greatly on pipelines. This year particularly marks a milestone for the pipeline industry as the ASCE Pipelines Division is becoming an institute. This will mean more opportunities for technical and personal growth, as well as more direct involvement of ASCE members in pipeline related activities.

In coordination with ASCE, the technical program and this publication was planned and implemented by the Technical Program Committee (TPC), which was led by the Technical Co-Chairs. We received a record high number of abstracts, and this turned the abstract selection process into a challenging task for the TPC. Nevertheless, the meticulous selection process based on a scoring system, followed by the TPC discussion on the abstracts with a critical score, resulted in a high quality technical program with seven tracks. These seven tracks, namely, Trenchless Installation, Design and Construction (2), Assessment and Rehabilitation (2), Operations, Maintenance, Risk and Safety, and Planning and Analysis, include 59 sessions of technical paper presentations, two panel discussions (one on fiber optics and one on energy generation in pipelines), and one session comprised of a presentation on engineering ethics by the ASCE. The overall technical program was further boosted by six workshops on pressure pipe design, large diameter pipes, specifications for cured-in-place pipe and manhole rehabilitation, asbestos cement pipe bursting, corrugated HDPE pipe, and AWWA pipe manuals of practice.

Our intent was to prepare a balanced technical program with today’s trends in mind for the pipeline industry, without sacrificing the overall quality of the content. We are also delighted to receive tens of abstracts from five continents; thereby, dubbing this event as an international conference.

On behalf of the Technical Program Committee, we are pleased to offer you the Proceedings of ASCE Pipelines 2015. We enjoyed reading these technical papers, and hope that you will find them useful and enjoyable too.

Warm regards,

Firat Sever, Ph.D., P.E., M.ASCE, and Lynn Osborn, P.E., M.ASCE
Technical Co-Chairs

Acknowledgments

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The Technical Program Co-Chairs thank the peer reviewers, most of whom also served as Session Moderators at the conference, for their timely reviews under a tight schedule and their leadership during the conference.

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The Technical Program Co-Chairs also thank the authors for their dedication to the industry in preparing a paper for this conference. Without your contributions, the conference would not be possible.

And lastly, the Technical Program Co-Chairs express special thanks to Ahmad Habibian and Camille Rubeiz, Conference Co-Chairs, for their vision and leadership during the planning and execution of Pipelines 2015, and also for their continual support and confidence in the Technical Program Co-Chairs.

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Sugarloaf Pipeline, Kp41 Tunnel—Design and Construction

Marcus Weeks[1]

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Abstract

The Sugarloaf Pipeline is part of the Victorian Government's strategy, "Our Water Our Future – The Next Stage of the Government's Water Plan". The strategy outlines infrastructure projects to address the worst year of drought and lowest stream flows in the State's history. The Sugarloaf Pipeline project involves the construction of a 70 km (43.50 mi) pipeline from the Goulburn River, located approximately 3 km (1.86 mi) north of Yea, to the Sugarloaf Reservoir at Yarra Glen. This paper presents the challenges associated with the design and construction of one of Australia's longest single drive pipe jack tunnels, the 828 m (2716.54 ft) long KP41 Tunnel. The tunnel was designed to avoid open excavation through the Toolangi State Forest where steep slopes, up to 40 degrees above the Melba Highway, would have required significant benching and excavation to facilitate construction. The tunnel was excavated using a Slurry Tunnel Boring Machine (TBM) at a continuous uphill grade of 0.5%. The tunnel was constructed with a two-pass tunnel lining consisting of a 2000 mm (78.74 in) ID reinforced concrete jacking pipe (200 mm (7.87 in) thick) primary lining and a 1750 mm (68.90 in) OD MSCL (Mild Steel Cement Lined) pipe (12 mm (0.47 in) thick, 19 mm (0.75 in) cement lining) secondary lining grouted in place.

1. PROJECT BACKGROUND

The Sugarloaf Pipeline is part of the Victorian Government's strategy, "Our Water Our Future – The Next Stage of the Government's Water Plan". The strategy outlines infrastructure projects to address the worst years of drought and lowest stream flows in the State's history.

Part of the water strategy is a $2 billion project to save water through the modernization of irrigation and other infrastructure in the Goulburn-Murray Region; Victoria's Food Bowl. The Food Bowl Modernisation Project is expected to deliver water savings of up to 225 GL annually in its first stage to be shared equally between the irrigation system, the environment and Melbourne. The Sugarloaf Pipeline and associated facilities, including a low lift pump station, balancing storage and high lift pump station, KP41 tunnel, inlet works and associated electrical infrastructure, transfers Melbourne's share of the water savings from the Goulburn River catchment near Yea to Melbourne's water distribution network via the Sugarloaf Reservoir.

The Sugarloaf Pipeline Project is being delivered by the Sugarloaf Alliance comprised of Melbourne Water Corporation, Sinclair Knight Merz Pty Ltd, GHD Pty Ltd and John Holland Group. The Alliance is responsible for planning and environmental assessments, engineering design, community and landowner consultation, project management and construction associated with the Project.

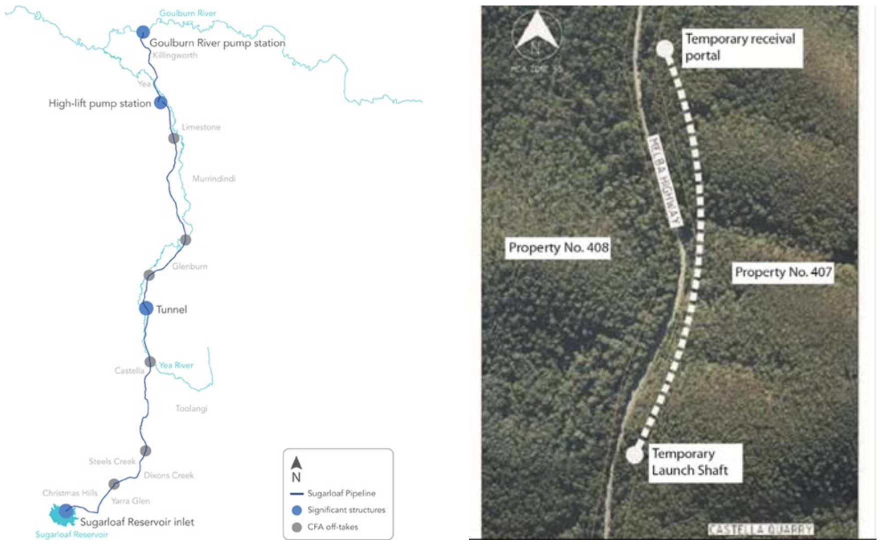
2. INTRODUCTION

The Sugarloaf Pipeline is generally aligned in a north south direction, and extends from the Goulburn River, located north of Yea, to the Sugarloaf Reservoir located at Yarra Glen. For the most part it follows the path of the Melba Highway along the Yea River valley, and the corridor is comprised predominantly of cleared agricultural land, state forest and rural living areas. The topography is undulating with steep, low-lying hills, gullies and waterways.

The KP41 Tunnel is located at approximately the 41 km (25.48 mi) point of the Sugarloaf Pipeline. This is within the northern section of the Toolangi State Forest, an established native forest that includes widespread large mature trees and significant ground covers, between the Old Castella Quarry to the south and Marginal Road to the North. The tunnel was designed to avoid open excavation through the Toolangi State Forest where steep slopes, up to 40 degrees above the Melba Highway, would have required significant benching and excavation to facilitate construction.

The tunnel alignment runs parallel to the Melba Highway and consists of a total length of 828 m (2716.54 ft), including a 680 m (2230.98 ft) long bend of 825 m (2709.97) radius designed to ensure a minimum of 15 m (49.21 ft) cover. Excavation was done using a Slurry Tunnel Boring Machine (TBM) at a continuous uphill grade of 0.5% from the launch shaft located at the south end of the alignment to the retrieval portal located at the north end of the alignment. Figure 1 provides an overview of the Sugarloaf Pipeline alignment and presents the alignment of the KP41 Tunnel.

Figure 1. (below) Pipeline Alignment and KP41 Tunnel



3. GEOTECHNICAL CONDITIONS

A number of site investigations were undertaken in a staged approach to assess the ground conditions for tunnelling. Initial investigations predominantly relied on field mapping, which were subsequently followed by a targeted program of exploratory borehole drilling and seismic geophysical surveys.

The investigations revealed two major geotechnical units along the tunnel alignment:

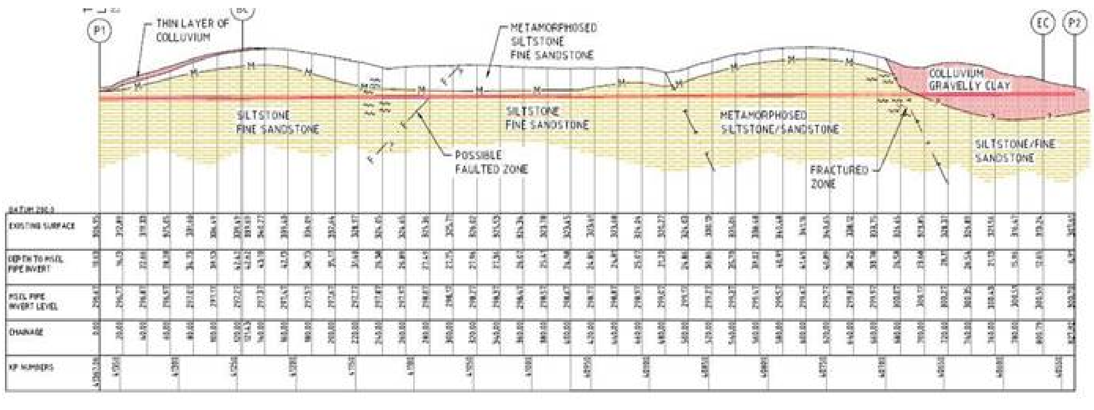
Unit 1 – Hornfels Rock: Consists of sedimentary rock belonging to the Lower Devonian Aged “Humevale Formation”, which has been locally metamorphosed to Hornfels. The Hornfels rock was interpreted to be present along approximately 85% of the tunnel alignment (chainage 0.0- 700 m (2296.60 ft)) and typically consisted of slightly weathered to fresh material with very high intact rock strength properties (150-200 MPa). The structure of the rock was quite blocky, containing 3-4 persistent planar joint sets, with little infill. The abrasive index of the rock was determined to fall mainly in the ‘Very Abrasive’ category (CERCHAR 2.0-4.0).

Unit 2 – Colluvium Material: At the northern part of the tunnel alignment a deep deposit of Colluvium resulting from an ancient landslip was identified. Colluvium was interpreted to cover approximately 15% (chainage 700-828 m (2296.60-2716.54 ft)) of the tunnel alignment. The Colluvium was typically ‘soil like’, being dominated by a reddish matrix of very stiff clay, with lesser amounts of rocky inclusions including gravel, cobble and angular boulder sized fragments.

Groundwater levels were approximately 20m (65.62 ft) above the tunnel at its deepest point.

Representative geological conditions of the two units are clearly visible in road cutting exposures along the Melba Highway and provided excellent ground condition information. Issues such as rock structure, the nature of the Colluvium/Hornfels interface and rock fragment particle size within the Colluvium were all clearly visible in the road cutting exposures.

Figure 2. (below) presents the interpreted geological long section along the KP41 Tunnel alignment.



4. DESIGN

4.1 Launch Shaft

The launch shaft for the KP41 Tunnel was located at the southern end of the alignment in a small, relatively flat area adjacent to the Melba Highway. It was located in a position to minimize the length of the tunnel, provide sufficient offset from the Melba Highway and to allow adequate working area for construction. The geotechnical conditions consisted of engineered fill overlying slightly weathered to fresh Hornfels. Due to the restricted site area, the shaft was designed to be rectangular with dimensions of 6 m (19.69 ft) wide, 10.6 m (34.78 ft) long and 11 m (36.09 ft) deep.

The temporary support for the shaft consisted of a reinforced concrete ring beam/retaining wall to support the construction surcharge loads and engineered fill followed by steel fiber reinforced shotcrete and patterned rockbolts to support the jointed rock mass. The design process for the temporary rock support consisted of the following:

- Defining the physical design parameters including construction surcharge loads.
- Calculation of the "Rock Tunnelling Quality Index" (Barton et al, 1974) using empirical analysis to determined the rock mass characteristics.
- Computer modeling of joint set data (Swedge). Swedge models the available site specific rock mass information to determine if there are any applicable failure modes not addressed by the empirical system. The computer modelling of

available joint set data, which involved a kinematic analysis, determines potential critical rock wedges and provides a basis for rockbolt length and spacing.

- Rockbolt design. Following an assessment of the potential critical rock wedge parameters and external load contributions, the rockbolt design was undertaken. Rockbolt design parameters including type, length, inclination, size/capacity and spacing were determined.
- Shotcrete design. The shotcrete design was completed in accordance with "Shotcrete Support Design in Blocky Ground: Towards a Deterministic Approach" (Barrett and McCreath, 1995).
- Computer modelling of the overall support system (Phase2).

The temporary rock support design consisted of 75 mm (2.95 in) thick steel fiber reinforced shotcrete and 3 m (9.84 ft) long, fully encapsulated, resin anchored rockbolts installed on a staggered 1.7 m (5.58 ft) grid.

Figure 3 (below) presents the temporary support for the shaft and the launch shaft site.

4.2. Retrieval Portal

The retrieval portal was located at the north end of the alignment adjacent to the Melba Highway. The geotechnical conditions consisted of colluvial material. The portal was cut into the side of a hill and the temporary support consisted of 150 mm (5.91 in) thick steel fiber reinforced shotcrete and 6 m (19.69 ft) long soil nails.

Figure 4 presents the retrieval portal site and the temporary support for the portal.

Figure 4. (above) Retrieval portal site and the temporary support

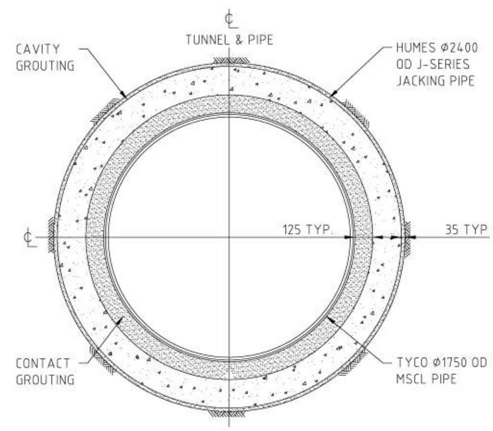
4.3. Tunnel Lining

4.3.1 *General*

The tunnel was constructed with a two-pass tunnel lining consisting of a 2000 mm (78.74 in) ID reinforced concrete jacking pipe (200 mm thick (7.87 in)) primary lining and a 1750 mm (68.90 in) OD MSCL(Mild Steel Cement Lined) pipe (12 mm thick (0.47 in), 19 mm (0.75 in) cement lining) grouted in place secondary lining.

A diagrammatic representation of the tunnel lining configuration is presented in Figure 5.

Figure 5. (below) Tunnel Lining



4.3.2 *Primary Lining*

The primary lining design was based on the horizontal alignment (drive length and radius of curvature), jacking and friction loads and for long term ground support. The design was generally in accordance with the "Guide to best practice for the installation of pipe jack and microtunnels" (Pipe Jacking Association, 1995) and "Pipe Jacking – Design Guidelines" (Concrete Pipe Association of Australasia, 1996). In addition to the above considerations the jacking pipes were designed using the following Australian, British and European standards:

- AS/NZS 3725-2007 Design for installation of buried concrete pipes. (Australian/New Zealand Standard, 2007)
- AS/NZS 4058-2007 Pre-cast concrete pipes (pressure and non pressure). (Australian/New Zealand Standard, 2007)
- BS 5911-1:2002 Concrete pipes and ancillary concrete products. (British Standard, 2002)
- EN 1916:2002 Concrete pipes and fittings, unreinforced, steel fiber and reinforced. (European Standard, 2002)

The KP41 Tunnel is one of Australia's longest single drive pipe jack tunnels. As a result of the tunnel length and radius of curvature, a key consideration for the design of the primary lining was an assessment of the forces required to jack the pipe. The jacking loads required to jack the jacking pipe were derived from the face load to advance the shield, self weight of the pipes in stable ground and friction around the pipes due to ground closure, misalignment and time delays

The component of friction around the pipe due to ground closure, misalignment and time delays is significant, but can be greatly reduced by the addition of bentonite lubrication through lubrication / grout ports installed in the jacking pipe. Past experience suggested that lubrication repeated every 2 – 3 days could reduce this component by more than 50%. In order to assess the jacking loads required to jack the pipe a frictional resistance of 1.25 kPa (0.18 psi) was adopted within the Hornfels and a frictional resistance of 5.0 kPa (0.73 psi) was adopted within the Colluvial material. These values were assumed based on published experience assuming bentonite lubrication was adopted.

The results of the assessment indicated that the maximum expected jacking load was 2750 tonnes (3031.37 tons), which was significantly higher than the maximum allowable jacking load specified for the jacking pipe. In order to reduce this load the KP41 Tunnel incorporated a series of interjack stations. Interjack stations were incorporated to limit the forces applied to the pipe and the thrust block wall by making use of the frictional forces induced by the trailing pipes. A total of 7 interjack stations were incorporated. The first interjack was installed 40 m (13.23 ft) behind the TBM and the subsequent 6 interjacks were installed at a spacing of 110 m (360.89). The combined jack capacity of the main jacks and interjack stations was designed to be well in excess of the maximum expected jacking load. It was considered that interjack stations were a cost effective risk mitigation measure due to the length of the tunnel, radius of curvature and the scale of any recovery works required should the jacking forces have exceeded the maximum allowable jacking load specified for the jacking pipe.

In addition to the forces required to jack the pipe, the primary lining was designed for a number of long term design loads including horizontal insitu stress capacity, radial loading from rock stress and radial loading from rock blocks.

Details of the primary lining jacking pipe are presented in Table 1.

Table 1 Primary Lining Properties

Primary Lining Property	
Length	3000 m (9842.52 ft)
Outer diameter (OD)	2400 mm (94.49 in)
Inner diameter (ID)	2000 mm (78.74 in)
Concrete compressive strength (F'c)	50 MPa (14 days)
Minimum cover to reinforcement	25 mm (0.98 in)
Maximum jacking force	1300 tonne (1433 tons) at 0.24 degrees
Steel collar band detail	8 mm thick mild steel (0.31 in)
Joint detail	Rubber ring
Design Life	150 years
Packer Type	MDF packer glued to pipe end
Packer Thickness	16 mm (0.63 in)
Approximate unit mass	11.0 tonnes (12.13 tons)

4.3.3 *Secondary Lining*

The secondary lining, 1750 mm (68.90 in) MSCL pipe, was designed for both internal and external hydrostatic loads. The internal hydrostatic load/pressure was consistent with the rest of the Sugarloaf Pipeline however the external hydrostatic load was significantly higher. The maximum external hydrostatic pressure, based on full hydrostatic load, was approximately 390 kPa. (56.56 psi) A design check was completed to ensure that this was below the critical buckling pressure for the 1750 OD MSCL pipe. The design check for the external hydrostatic pressure was based on the Jacob paper "Buckling of Circular Rings and Cylindrical Tubes Under External Pressure", (ASCE Manuals and Reports on Engineering Practice No. 79 Steel Penstocks – Section 6 – Steel Tunnel Liners, 1993).

An additional key consideration for the design of the secondary tunnel lining was durability and corrosion. The corrosion mitigation strategy for the tunnel was to provide an equivalent asset life to that of the buried pipeline (150 years). The preferred strategy was to install an uncoated MSCL welded pipeline within the jacking pipe and grout this in place with cementatious grout. This would provide a high pH/alkalinity environment against the steel surface equivalent to the cement lining of the pipeline, effectively inhibiting corrosion of the steel pipe. The key requirement was the control and verification of the grouting process to ensure that the grout had completely filled the annulus between the 1750 mm (68.90 in) MSCL pipe and the jacking pipe.

5. CONSTRUCTION

5.1 TBM Selection

A slurry type TBM was adopted for the tunnel excavation as it had to cope with all of the expected geological conditions including high strength rock, potentially high groundwater inflow, the possibility of large boulders within the Colluvium and the hard-to-soft ground interface zone. The excavated diameter of the TBM was 2475 mm (97.44 in) and the cutterhead design consisted of a hard rock head with 12 inch rings (6 x singles, 7 x double). The main jacking station thrust had a capacity of 1400 tonnes (1543.24 tons) and the interjack stations had a thrust capacity of 1000 tonnes (1102.31 tons).

5.2 TBM Utilization and Advance Rates

TBM utilization and advance rates varied dramatically between excavation in the rock and excavation in the colluvium. Figure 6 presents the TBM utilization in rock and colluvium.

The excavation through the rock was characterized by long cutting times. For the most part, and excluding pockets of softer rock, excavation proceeded at 20 – 25 mm/min (0.79 – 0.98 in) throughout each 3 m (9.84 ft) pipe. This resulted in an excavation cycle time of between 2.5 – 3 hrs including the racking of a new pipe. The STP was run with water as the spoil transport medium and the centrifuge was able to keep density under control without the use of flocculent. Cutterhead maintenance was required regularly, twice per week, to change ground engaging tools. Surveyors required a stoppage in excavation every 60 – 80 m (196.85 – 262.47 ft).

Excavation through the colluvium was characterized by long stoppages to allow the STP to treat the slurry. Excavation rates were between 100 – 150 mm/min (3.94 – 5.91 in), resulting in 3 m (9.84 ft) pipes being excavated in 25 minutes. Racking the new pipe took 15-20 min followed by an average 2 – 2.5 hrs delay for the STP to control the slurry density and viscosity.

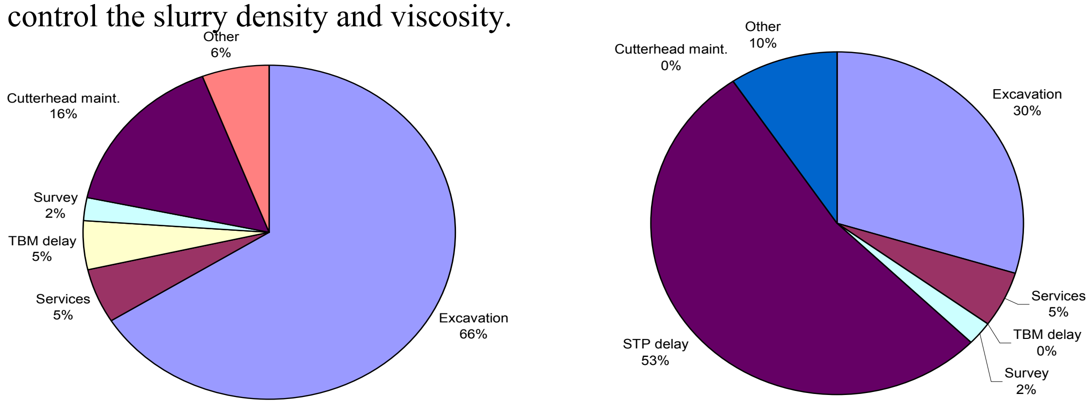


Figure 6. TBM Utilization in rock and colluvium

Overall production rates were high and downtime was kept to a minimum. This was achieved as a result of strict adherence to the machine maintenance requirements and a suitably anticipated on site spares store. Production rates for the tunnelling works averaged approximately 60 m (196.85 ft) per week with a maximum production rate of 118 m (387.14 ft).

5.3 Observed Jacking Loads and Frictional Resistance

The jacking loads and frictional resistance were estimated from the total thrust force measured at the main jacking station in the shaft and the cutterhead force measured at the steering cylinders on the machine. The difference between the two is the approximate friction force on the pipe string. The jacking loads and frictional resistance is presented in Figure 7.

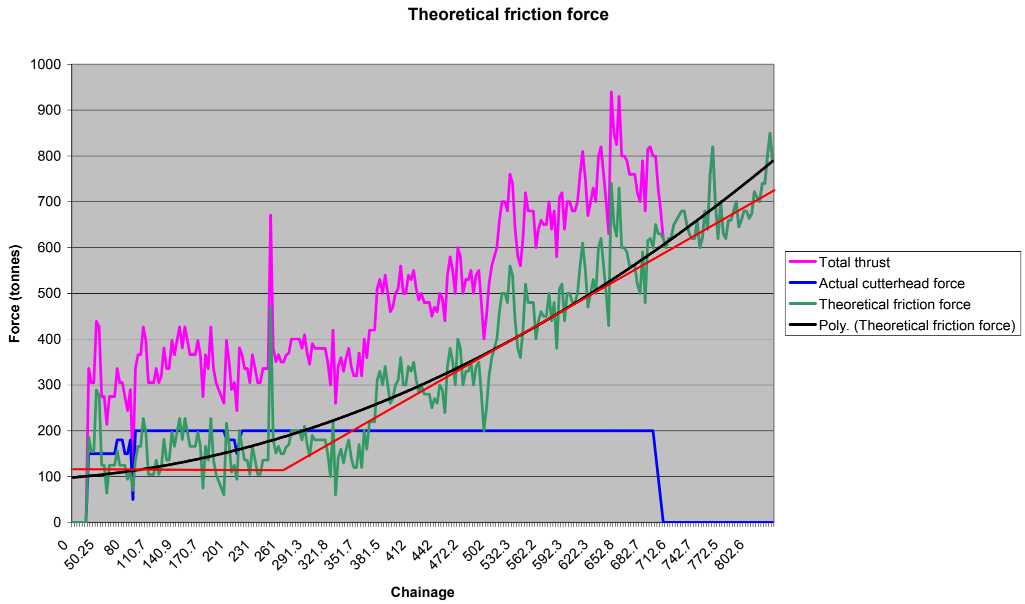


Figure 7. Friction Force

As expected, the trendline indicates that the friction force increases with the length of the tunnel. The maximum jacking load required to jack the jacking pipe was 950 tonnes (1047.2 tons). This was significantly lower than the maximum expected jacking load of 2750 tonnes (3031.36 tons) and as a result no interjack stations were operated during excavation of the tunnel.

5.4 Tunnel Survey

Survey for a curved pipe jacking operation is unusual in that it must continuously and accurately maintain the machine's position whilst the machine, survey equipment and reference prisms are all moving with the pipe string. For the KP41 Tunnel the long drive length and curved alignment added the complication that a sharp correction or unnecessary variation in excavated alignment could add significantly to the total thrust forces required.

The survey equipment and software utilized was a VMT SLS-RV system which was maintained and operated by a local specialist survey contractor. The theoretical assumption that allows the software to function is based on the mathematical assumption that the location of each of the survey elements is equal to the excavated location at their respective chainages.

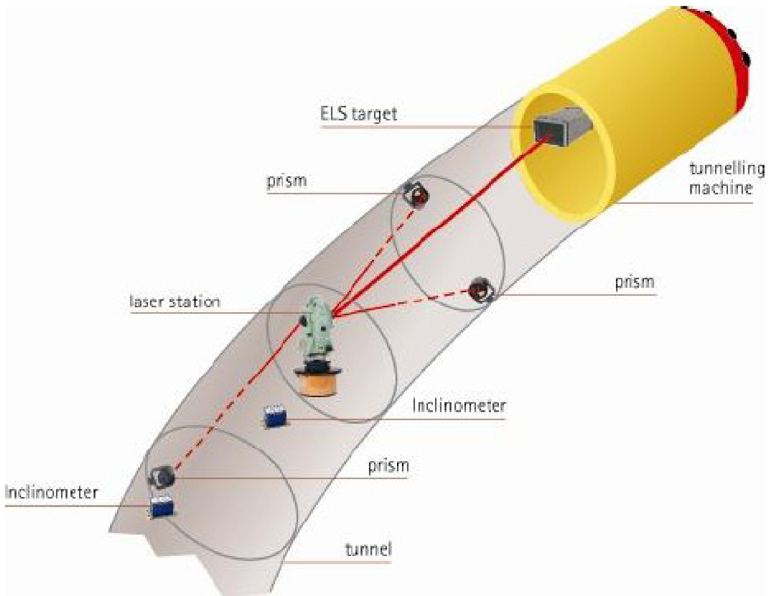
Survey for curved pipe jacking is generally described as the following three phases:

- Phase 1 - Theodelite solidly based in shaft directly measuring ELS target
- Phase 2 - Theodelite established in jacking pipe but still able to backsight prism in shaft

- Phase 3 - Theodelite and backsights all travelling with the pipe string

Once phase 3 begins, the excavation must be stopped every 60 – 80 m (196.85 – 262.47 ft) to re-establish a known location for the theodelite. This is done conventionally by traversing from the shaft to the machine. Every 400 m (1312.34 ft) a specialist gyroscopic survey was conducted to apply a very accurate check of location.

Figure 8. presents the basic set up of the survey equipment.



5.5 Observed Rock Strength Values

Rock strength was monitored during tunnelling within the sedimentary rock belonging to the Lower Devonian Aged "Humevale Formation" (Hornfels). The rock strength was typically between 150 MPa and 200 MPa, which was consistent with the range of UCS values determined during investigations from laboratory UCS testing. A drop in rock strength between chainages 400 and 550 m (1312.34 – 1804.46 ft) was observed in both the investigation and construction period. The tunnel advance rate was noted to have accelerated from 80 m/week (262.47 ft/week) to 120 m/week (393.70 ft/week) within this zone

The tunnel face was inspected once during tunnelling operations at chainage 576 m (1889.76 ft). Joint spacing was in the order of 200 mm to 300 mm (7.87 – 11.81 in), with a maximum of 500 mm (19.69 in) observed. The joints were tight, but showed signs of joint alteration, including weathering and iron staining. Three dominate joint sets were persistent in the rock structure, including two joint sets dipping steeply (70-80 degrees) to the north west and north east, and a third dipping at 45 degrees to the south. A fourth random joint was also observed, and was noticeably more weathered than the three principal joint sets. The jointed nature of the rock mass played an important role in achieving relatively high production rates as the TBM cutters were able to dislodge rock from the face along existing defect planes much more readily than what would have been able to be achieved in a more massive (unjointed) rock formation. Figure 9 presents the tunnel face at chainage 576 m (1889.76 ft).

Figure 9. (above)Tunnel Face CH 576 m

5.6 Groundwater

Groundwater levels were monitored during construction both at the launch shaft, retrieval portal and along the tunnel alignment. At the launch shaft the groundwater was approximately 3 m (9.84) below ground level (BGL), and was dropped to 11.5 m (37.73) BGL to facilitate construction. Dewatering rates estimated from slug testings' completed during design investigations predicted a daily average dewatering rate in the order of 30 m^3/day (98.43 ft^3/day) to 60 m^3/day. (196.85 ft^3/day) During shaft construction dewatering rates reached 60-70 m3/day as the shaft approached full depth.

During tunnelling the dewatering rate was consistently in the order of 60 m^3/day. (196.85 ft^3/day) However, days of increased dewatering rates were observed, typically when groundwater was allowed to drain into the tunnel face during cutter changes. This trend was reflected in a localized drop in standing water levels along the tunnel alignment in the order of several meters. These drops stabilized as the TBM passed, and are currently showing signs of recovery.
Groundwater quality was monitored during construction and the results indicated that the groundwater was suitable to be used within the Sugarloaf Pipeline construction corridor for dust suppression.

5.7 Secondary Lining

In order to install the 1750 mm (68.90 in) MSCL secondary lining, a pipe carrier was designed to install each pipe section to its nominated location within the tunnel, join it and then return to the retrieval portal to collect the next pipe. The pipe carrier was designed to:

- Lift and place a full length (13.5 m (44.29 ft)) MSCL pipe weighing 11 tonnes (12.13 tons) both on the surface and in the tunnel
- Negotiate the curvature of radius of the alignment by a self levelling/self-steering hydraulic mechanism
- Provide fine adjustment horizontal and vertical movement of both ends of the pipe within the tunnel to allow it to be centralized and supported

Figure 10. (left) presents a view of the pipe carrier located at the Retrieval portal.

6. CONCLUSIONS

One of the key design issues for the KP41 Tunnel was an assessment of the forces required to jack the pipe. The jacking loads required to jack the jacking pipe are derived from the following components:

- Face load to advance the shield
- Self weight of the pipes in stable ground
- Friction around the pipes due to ground closure, misalignment and time delays

The results of the assessment indicated that the maximum expected jacking load was 2750 tonnes (3031.36 tons) which was significantly higher than the maximum allowable jacking load specified for the jacking pipe. A total of 7 interjack stations were incorporated in the design. During construction the maximum jacking load required to jack the jacking pipe was 950 tonnes (1047.2 tons) and as a result no interjack stations were operated during excavation of the tunnel. Whilst no interjack stations were used during tunnelling, they provided a cost effective risk mitigation measure due to the length of the tunnel, radius of curvature and the scale of any works required should the jacking forces have exceeded the maximum allowable jacking load specified for the jacking pipe.

Construction of the launch shaft, retrieval portal and primary tunnel lining was completed in July 2009. Whilst TBM utilization and advance rates varied dramatically between excavation in the rock and excavation in the colluvium, overall production rates were high and downtime was kept to a minimum. Observed geotechnical conditions and rock strengths were generally consistent with those predicted during initial geotechnical investigations. Installation of the secondary lining is ongoing.

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Challenges and Rewards of a Successful Compound Curve Microtunnel Drive

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Abstract

The Santa Ana River Interceptor (SARI) Line is a 23-mile-long wastewater pipeline that extends from the boundary of Orange and San Bernardino counties in Southern California to the Orange County Sanitation District (OCSD) sewage treatment plant. Several segments run adjacent to and under the Santa Ana River and nearly 4 miles of the pipeline were in jeopardy of failure during heavy rainstorms. To ensure the long-term integrity of the pipeline, protect public health and safeguard the environment, the Orange County Flood Control District, in cooperation with other stakeholders, embarked on the Santa Ana River Interceptor Relocation Project to relocate and replace the SARI Mainline with a new pipeline. A portion of the project consisted of 4,000 feet of 101.5-inch OD microtunnel completed in four drives including installation of 2,900 feet of 84-inch ID reinforced concrete pipe and 1,100 feet of 99.5-inch ID steel casing. This design also required excavation of several deep shafts in difficult locations. Ground conditions along the four alignments were an extremely abrasive mixed face combination with soft to stiff silt and loose sand, gravel, sand, and clay exhibiting a flowing behavior with cobbles and some boulders. The groundwater table ranged from the tunnel invert at the lowest point to approximately 17 feet above the tunnel invert at the highest. Originally designed as a traditional conservative microtunnel project with several short, straight drives, the SARI Mainline offered an ideal opportunity for an innovative value engineering proposal. By suggesting a standard curve microtunnel drive and a compound curve microtunnel drive utilizing an innovative hydraulic joint the project contractor was able to eliminate shafts and combine multiple drives on the project. Reinforced Concrete Pipe (RCP) with Carnegie style bell and spigot joints was designed for use on the sections of the project being considered for the curved drives. After revising the design and incorporating appropriate changes to the standard pipe, the RCP manufacturer concurred that the pipe would perform through the curves, an important step in the value engineering ("VE") process. Preliminary sketches of a curved alignment were prepared to determine the potential curve radii. Based on the sketches and load calculations it was determined that RCP with the incorporation of the hydraulic joint was more than sufficient to handle the potential joint deflection. The coordination and cooperation between the agency, contractor and manufacturer resulted in a savings to the project of over $1 million and offered a 20% reduction in the tunneling schedule.

Introduction

Microtunneling is inherently a challenging form of trenchless construction –monitoring jacking forces as the pipe string moves, balancing the slurry at the face of the microtunnel boring machine and completing the drive within the tolerances identified by the contract create challenges in the best of conditions. When extremely abrasive soil conditions, site constraints and environmentally sensitive areas are factored in, the project becomes formidable.

The SARI Project

The Santa Ana River Interceptor (SARI) Line is a 23-mile-long wastewater pipeline that extends from the boundary of Orange and San Bernardino counties in southern California to the Orange County Sanitation District (OCSD) sewage treatment plant (Figure 1). Several segments run adjacent to and under the Santa Ana River and nearly 4 miles of the pipeline were in jeopardy of failure during heavy rainstorms. To ensure the long-term integrity of the pipeline, protect public health and safeguard the environment, the Orange County Flood Control District, in cooperation with other stakeholders, embarked on the Santa Ana River Interceptor Relocation Project to relocate and replace the SARI Mainline with a new pipeline (Figure 2).

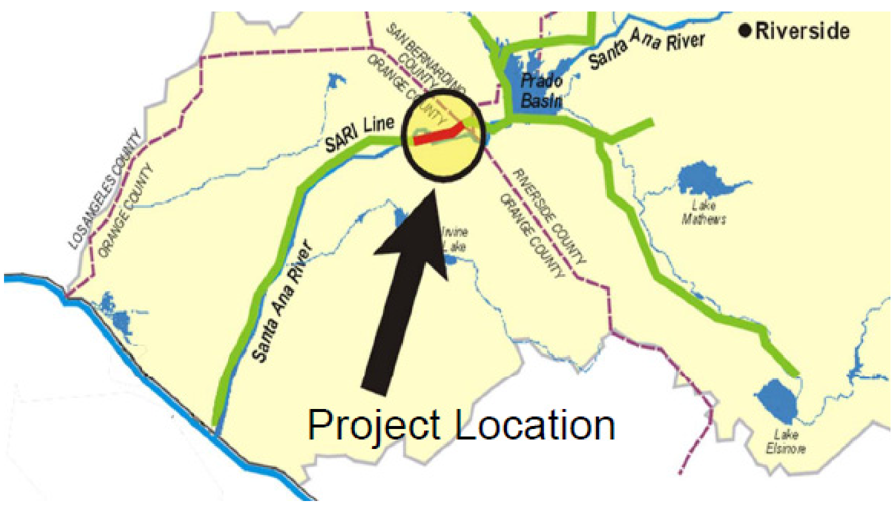

Figure 1
Project Location

Figure 2
Overall Project Alignment

The Orange County Public Works Department (OCPWD) recognized that a qualified contractor with the proper microtunnel equipment and experience was required for the project. SARI project specifications required a highly specialized microtunnel boring machine with an airlock and compressed air access to the face of the machine to cope with the expected soil conditions. James W. Fowler Co. (JWF) was ultimately named the County's choice for the microtunneling portion of the project.

Due to location of the project and its proximity to the 91 Freeway and Santa Ana River, there were a number of stakeholders with a sizeable investment in the successful completion of the project. The project team included Orange County Public Works Department, Orange County Sanitation District, Orange County Flood Control District, Orange County Parks, Orange County Watershed, US Army Corps of Engineers, California Department of Fish and Game, Santa Ana Water Projects Authority, Yorba Linda Water District, City of Yorba Linda, City of Anaheim, Canyon RV Park, Green River Golf Club and California State Parks.

Project Details

A portion of the SARI project consisted of 4,000 feet of 101.5-inch OD microtunnel done in four drives with installation of 2,900 feet of 84-inch ID reinforced concrete pipe and 1,100 feet of 99.5-inch ID steel casing. This design also required excavation of several deep shafts in difficult locations. Ground conditions along the four alignments were an extremely abrasive mixed face combination with soft to stiff silt and loose sand, gravel, sand, and clay exhibiting a flowing behavior with cobbles and some boulders. The groundwater table ranged from the tunnel invert at the lowest point to approximately 17 feet above the tunnel invert at the highest.

The original design of the SARI Mainline included a typical microtunnel project with seven straight drives of lengths ranging from 120 to 1,190 feet. The use of Jackcontrol hydraulic pipe joints provided an opportunity to introduce curved microtunnels into the project alignment, reducing the mobilization of equipment and eliminating several shafts required to build the project, including one that would disrupt a recreational vehicle park.

The contractor proposed four drives – two straight, one compound curve and one standard curve. The first drive, approximately 620 feet, was located under Gypsum Canyon Road and Gypsum Canyon Drainage channel. The second drive was 1,089 feet under a portion of the Green River Golf Course and the Santa Ana River channel. These two drives were not connected and curves offered no advantages.

The final two drives adjacent to the 91 Freeway fit the profile for adding curves. The third drive, the compound curve, was adjacent to the 91 Freeway and was approximately 1,567 feet. This drive was the first use of the Jackcontrol AG hydraulic joint system with real-time monitoring in North America and set a United States record for being the longest microtunnel compound curved drive and only the second compound curve drive in the United States. The final drive of 622 feet was also adjacent to the 91 Freeway and included a standard curve utilizing the Jackcontrol system (Figure 3).

Figure 3
Microtunnel Alignment

Value Engineering Proposal

Originally designed as a traditional conservative microtunnel project with several short, straight drives, the SARI Mainline presented the contractor an ideal opportunity to offer an innovative value engineering proposal. By suggesting a standard curve microtunnel drive and a compound curve microtunnel drive utilizing an innovative hydraulic joint manufactured by Jackcontrol AG of Switzerland and a VMT GmbH theodolite-guided navigation system, the contractor was able to eliminate shafts and combine multiple drives on the project.

A meeting was held between the contractor and the hydraulic joint manufacturer to review and evaluate the potential curved drives. It was determined that sections of the project could benefit from a redesign. Reducing the five straight drives with the corresponding three jacking and three reception shafts to two longer curved drives requiring two jacking shafts, one reception shaft and an observation shaft would meet the intent of the original design. An added advantage to the revised alignment was the option to adjust the location of the shafts, selecting areas that had better access and site availability.

Reinforced Concrete Pipe (RCP) with Carnegie style bell and spigot joints was determined to be the ideal product for the curved drives and Ameron Water Transmission Group was selected as the pipe manufacturer. After revising the design and incorporating appropriate changes to the standard pipe, Ameron concurred that the pipe would perform through the curves, an important step in the value engineering ("VE") process. Preliminary sketches of the revised alignment were prepared to determine the potential curve radii and it was confirmed that RCP was more than sufficient to handle the potential joint deflection.

Once the contractor and manufacturers were convinced that the curved drives could be completed successfully and that the project could benefit from the introduction of curved microtunnels, a preliminary VE proposal was submitted to the project stakeholders and their engineers. It was important to gauge their acceptance of incorporating curved drives into the project since at that time there had only been one curved drive completed in the United States. The project team was very enthusiastic about the possibility of eliminating several of the tunnel shafts and the potential corresponding reduction in the tunnel and shaft construction schedule.

While the Owner was receptive to the potential VE proposal, they chose to reserve final determination until the cost and schedule savings could be compared against the added risk that a curved drive would introduce into the project. The dollar values of the changes were analyzed and the schedule was revised. The contractor determined that 30 days could be saved on the tunnel schedule by incorporating the curved drives and since the tunnel was in the project critical path, that translated directly to 30 days savings to the project schedule. In total the VE proposal was estimated to save the project over $1 million and offered a 20% reduction in the tunneling schedule.

With the VE proposal approved, the contract drawings were revised to include an alignment that would produce a buildable tunnel and stay within the project's permanent easement, a process that was not nearly as easy as it may seemed.

Project Partners Contribute to the Successful Curved Drives

Pressure transmission rings made of wood material are widely utilized in microtunneling applications. In curves, the mechanical characteristics of wood material can cause severe damage to jacked pipes. To avoid such damage on the planned curved drive, the contractor partnered with the manufacturer to use a patented hydraulic joint and real-time monitoring system. The joint acts hydrostatically like a fluid-filled hose with a uniform pressure level allowing the curved joints proposed on the project without causing axial stresses that would exceed the strength of the pipe material (Figure 4).

Figure 4
Photo of Hydraulic Joint

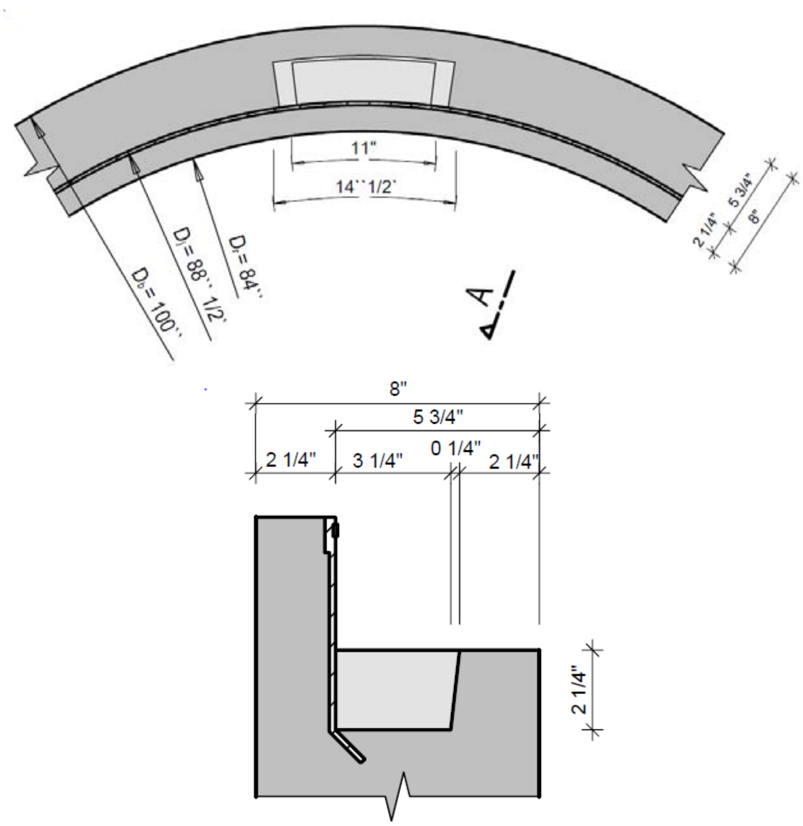

Figure 5
Joint Detail

The jacking loads were also examined in relation to the pipe joint mating surfaces. Here the hydraulic joints proved to be the perfect alternative for increasing the contact area of the jacking surfaces in the curves. This provided the jacking forces needed to complete the drives without point loading the pipe joints. The bells on RCP were manufactured with a blocked out area to accommodate the hydraulic joint requirements (Figure 5).

Based on the mechanical characteristics, the hydraulic joint allows curved microtunneling alignments with the use of regular pipe lengths and application of usual jacking forces without harming the pipe structure. The hydraulic joint, with its well-defined and reversible mechanical characteristics, was used as an integrated sensor for a reliable determination of the size and position of the thrust/resulting jacking force during the jacking operation. This capability provided a real-time monitoring of the pipe structure regarding the admissible jacking force to prevent the pipe from being damaged.

The tunnelling machine utilized was a Herrenknecht AVND 2000D equipped with a SLS-Microtunnelling LT navigation system. The navigation system was chosen for the guidance of the long distance and curved pipe jacking application on the SARI project. The main component of the system was a servo motorized laser total station mounted inside the tunnel on a special bracket which moved together with the pipeline. The actual position of the laser total station was continuously calculated with help of the known as-built position of the already installed pipes.

An experienced tunnelling engineer was on the jobsite for the duration of the compound curve drive and worked closely with the contractor and performed all of the control surveys in the tunnel and on the surface. The engineer conducted daily control measurements as required by the Owner and provided reports to confirm that the position of the machine and pipes were within the specified tolerances. Another value offered by the system was the ability to access the survey data from anywhere in the world through a web-based interface. The contractor was able to log into the website and monitor the machine progress, location and alignment instantly.

A Successful Completion

The ultimate success of the project was threefold: The agency was open to the innovative curved drive proposal offered by the contractor, the hydraulic joint and guidance system ensured the drives were completed as designed and the relationship created by the project team developed into a true partnership where the project success came first.

The accomplishments on the SARI project, and the two curved drives in particular, were a testament to the planning that occurred prior to the project, the vetting of the microtunneling contractor through a rigorous prequalification process and the commitment of the team in ensuring the project was successful.

Microtunneling Technology Implemented for the Replacement of an Aging One Mile PCCP 36-inch Force Main to Minimize Environmental Impacts

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Abstract

In 1994, 2001, and 2009 Fairfax County completed spot inspections and testing (i.e. visual, petrographic examinations, wire testing for tensile and torsional strength) at different locations along an existing one mile 0.91 m (36-inch) PCCP which traversed an environmentally sensitive area. The purpose of the testing and evaluation was to assess the pipe integrity and determine if replacement or repair was required. The results of these tests indicated that the pipeline remained in serviceable condition. However, concern regarding any extended leakage into the surrounding environment led to a decision for installing a new, parallel 0.91 m (36-inch) pipe, with the existing line to be available for redundancy. Due to the proximity to natural wetlands open trench installation was not plausible; therefore, the use of trenchless technologies was evaluated for its viability in this application. Based on the analysis, it was determined that microtunneling (MT) was the most viable technology for installing the new force main in order to minimize environmental impacts to the wetlands and disturbance to the neighboring communities. Microtunneling proved to be the best available technology in this application as the new ductile iron force main has been in operation for two years and had minimal impact on the wetlands and neighboring communities.

INTRODUCTION

Aging infrastructure, such as force mains and pipelines, is a concern for many utilities due, in some cases, to their inability to cost-effectively replace these force mains by conventional (open trench) means. Areas that were once open fields or roads are now highly populated regions that require more advanced approaches for installing/replacing force mains. These more advanced approaches include trenchless technologies of which many public utilities are now considering in order to avoid existing utilities/structures and natural habitats. Additionally, Utility Owners today have a much greater understanding of the significance of environmental impacts that open cut trench construction of force mains can have on the environment. The increased use of trenchless technologies, especially microtunneling, is becoming increasingly more viable to Utility Owners to minimize impacts to the environment and disruptions to the public.

Initial construction of the 0.91 m (36-inch) Dogue Creek Force Main was in 1977 and PCCP pipe was used for the force main. Prestressed wire manufactured for pipe during this era frequently included "Class IV" wire, which has had a questionable performance record. This paper summarizes the methodical approach that was used in order to monitor an existing 0.91 m (36-inch) force main integrity, the process of evaluating trenchless technologies, design of the selected trenchless technology, and addressing construction related issues which includes commissioning of the new force main.

EVALUATION AND DESIGN PHASE

Existing Force Main Integrity

Understanding the integrity of the existing 0.91 m (36-inch) force main was the first critical step as it helped in the planning process for implementing the appropriate studies necessary, design of the new force main, and account for the construction phase of the project in the scheduling of the work. A condition assessment was implemented to evaluate the force main integrity which consisted of both visual and physical testing of the PCCP force main. The original condition assessment was completed in 1994 with follow up assessments performed in 2000 and 2009. Each condition assessment consisted of pipe sounding, visual inspections, selected removal and testing of pre-stressed wire and mortar coating analysis. Table 1 provides a summary of the specific testing performed as well as the purpose of each test.

Table 1. Condition Assessment Analysis of PCCP

Test	Purpose
Pipe Sounding	Determine if delamination is occurring in the pipe
Visual inspections	Assessment for visual cracks or other defects
Removal of prestressed wire	Test the wire for torsion, tensile and embrittlement properties
Removal of mortar coatings	Perform petrographic analysis to ascertain condition of mortar
Soil testing	Analyze soil pH, corrosivity, etc.

Based on the results of the testing performed, it was determined there were no visible signs of distress in the pipe nor did it appear that delamination was occurring. The quality and condition of the mortar coatings tested were sound and appeared to be providing the necessary protection for the underlying steel cylinder and prestressed wires. The prestressed wire appeared to be a better quality than “Class IV”. However, it was noted that there was minor corrosion visible on the steel can as well as on several of the pre-stressed wires. This corrosion was likely attributed to concentration cells produced by differences in local areas of the mortar coating and wire surface. The conditions contributing to the development of concentration cells are differences in porosity, moisture, oxygen or pH in the adjacent soil areas (Padewski, 2009)

Additional testing of the interior of the force main was considered; however, it could not be taken out of service due to it being the only means for conveying sewage from this portion of the County to the wastewater treatment plant. Hence a visual inspection of the interior of the force main was not possible. Other interior testing, such as acoustic monitoring, was deemed unnecessary.

Based on the testing and analysis performed and risk factors associated with the local environmental sensitivity and difficulty of accessing any future leak, Fairfax County concluded that paralleling the existing force main was the best approach to implement in their capital improvements program. The existing PCCP force main would be utilized to provide redundancy which the County currently does not have for this portion of the County's system.

Evaluation of Trenchless Technologies

As previously noted the original construction of the 0.91 m (36-inch) Dogue Creek Force Main was in 1977 using open cut type construction technique which traverses wetlands, streams, residential properties and the Fort Belvoir Military Base. Due to permitting requirements, regulations, the development of the properties in the nearby areas, and the Fort Belvoir Military Base requirements, open cut construction was not a viable option for this force main replacement. Specifically, 84% of the force main needed to traverse the Fort Belvoir Military Base, and it was determined that open trench construction would be too disruptive to their daily operations. Approximately, one third of the force main crossed wetlands or streams which was not conducive to open trench installation of the force main. Additionally, the force main crossed a state highway which would also not allow open trench installation. As a result, alternative methods such as trenchless technologies needed to be considered for the installation of the new force main.

With the decision to proceed forward with trenchless technology for installation of the new force main the next question to address is which technology would be best suited given the field constraints. Based on the conditions developed for this project, it was determined that horizontal directional drilling (HDD), microtunneling (MT) and horizontal earth auger boring (HEB) were the trenchless technologies to be further considered for this application. Table 2 provides a general summary of the key attributes of each method as well as its benefits over the other technology.

In summary, the HDD trenchless method has good installation accuracy as well as the ability to have long drive spans when compared to the other alternatives. On the downside the HDD method requires a larger foot print due to the need to "string" out of the pipes during installation which in turn creates more ground disturbance resulting in potential impacts to the environment and Ft. Belvoir operations. MT offers minimal disturbance due to the relatively small shaft sizes in comparison to HDD and HEB. Launching and receiving shafts can be strategically positioned along the alignment of the old force main to minimize disturbance to Fort Belvoir operations as well as the environment by avoiding the wetlands and other natural habitats. HEB is often been used for straight, short drives (e.g. under a roadway) in relatively stiff or dense soils above the high groundwater table. Given the nearby wetlands and groundwater table elevations on this project as well as the lack of control over the soils at the face of the machine, HEB was not suitable for this application.

Table 2. Trenchless Technology Methods Comparison

Evaluation Criteria	Trenchless Technology		
	Horizontal Directional Drilling (HDD)	**Microtunneling (MT)**	**Horizontal Earth (Auger) Boring**
Accuracy of alignment	Accurate Drives	Very Accurate Drives	Not Accurate over longer runs
Suitable Drive Span or Tunneling Length	6.1 m - 1,524 m (200 ft. - 5,000 ft.)	30.5 m – 152.4 m (100 ft. - 500 ft.)	18.3 m – 121.9 m (60 ft. - 400 ft.)
Casing/Pipe Material	High Density Polyethylene (HDPE)	Prestressed Cylinder Concrete Pipe (PCCP), Glass Fiber Reinforced Plastics (GRP, Steel, Ductile Iron, and other options	Prestressed Cylinder Concrete Pipe (PCCP), Glass Fiber Reinforced Plastics (GRP, Steel, Ductile Iron, and other options
Shaft Space Requirements Comparison	Large Foot Print	Small Foot Print	Smaller Foot Print

Based on considering the benefits of each technology as well as the potential risks of implementing each one for the project, it was determined that MT would be best suited for the new 0.91 m (36-inch) Dogue Creek Force Main.

Development of the Tunneling Concepts and Design

With the establishment of MT as the best available technology for this project, the next step in the conceptualization/design process was to develop an alignment and identify the number of shafts needed based on the suitable shaft drives for MT as well as considering the above grade obstacles that may interfere with the installation of the launching or receiving shafts. Figure 1 shows the general alignment that was determined to be the most viable option for the MT process. It consisted of six shafts ranging in depths from 9.75 m (32 ft.) to 15.85 m (52 ft.).

After the alignment was finalized, based on the technical requirements, it was thoroughly reviewed by Fort Belvoir, the Virginia Department of Transportation and other regulatory agencies in order to coordinate the construction activities within the base and address any environmental impacts.

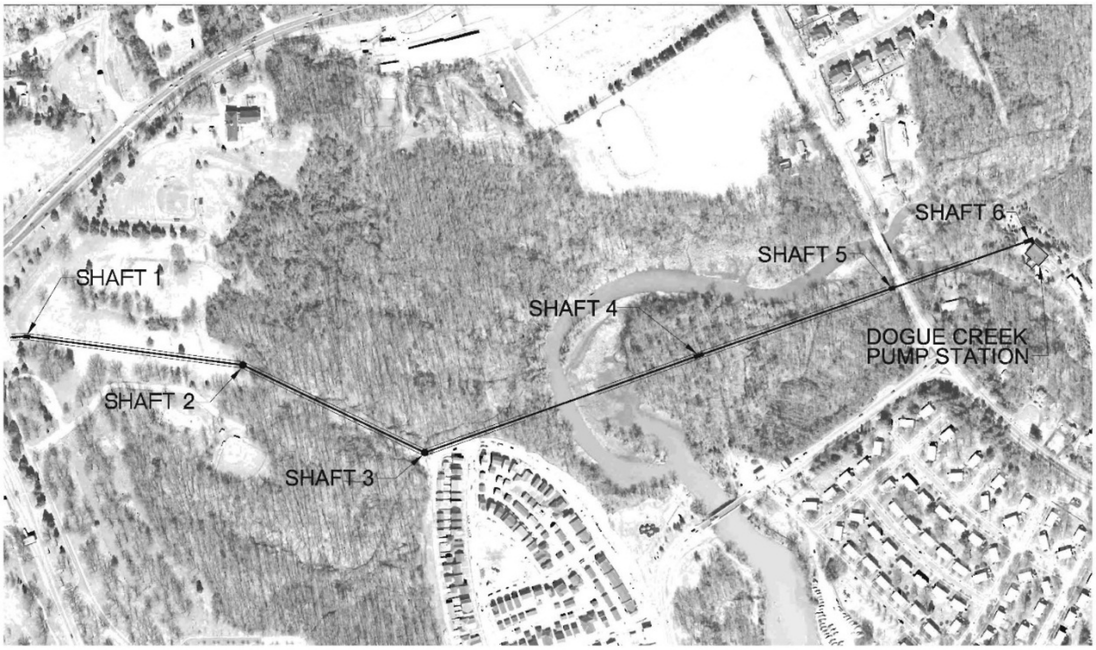

Figure 1. Microtunneling General Alignment

Once the approval of the alignment by all the key stake holders was received, the design team proceeded with developing the details of the design which included the following items:

- Shaft design details based on subsurface conditions
- Analysis of dewatering requirements for each of the shaft sizes
- Ground and vibration monitoring plans
- A bentonite “frac out” plan
- A hydraulic analysis for the pumping surge on the Dogue Creek Pump Station

As previously mentioned, there were a total of six launching and receiving shafts. In some cases one shaft would serve as both a launching and receiving shaft. The shaft design varied for each location and was based on the subsurface conditions. Generally, three main designs were used and consisted of solder piles and wood lagging, sunk in caissons, and sheet piling. Table 3 provides a summary of the design for each shaft.

Each shaft was also constructed with a concrete floor mat to provide the contractor a stable working surface for the tunnel operations.

Dewatering Considerations

Dewatering was also a concern during design due to the proximity of the shafts to wetlands and nearby streams. This was especially true for shafts 4 and 5 which were located next to a wetlands/marsh area as well as close proximity to Dogue Creek. In order to better understand the dewatering requirements at each shaft, a two phased approach was used analyze the anticipated water infiltration. Phase I consisted of using Plaxis Flow modeling to calculate inflow requirements using empirical methods while

Table 3. Microtunneling Shaft Design

Shaft Number	Shaft Design	Shaft Depth
1	Solder Piles and wood lagging	9.75 m (32 ft.)
2	Sunk-in Caissons	12.19 m (40 ft.)
3	Sunk-in Caissons	15.85 m (52 ft.)
4	Sheet Piling	11.89 m (39 ft.)
5	Sheet Piling	10.36 m (34 ft.)
6	Sheet Piling	10.97 m (36 ft.)

Phase II would validate the model by performing actual field test to confirm the inflow rates calculated in the model.

Phase I (Plaxis Flow modeling) began with developing a subsurface condition profile within the influence zone of each shaft to simulate the ground conditions that would be included in the finite element model. Each shafts excavation support system was then modeled to determine its influence on the ground water inflow volume at each shaft. The model incorporated various permeability values for the soils in an attempt to assess the potential high and low volume of anticipated ground water volume.

Table 4 summarizes of the Plaxis Flow modeling results for each shaft. In reviewing the data, the model estimated steady state flows ranging from 27.3 m^3/d (5 gpm) to 490.6 m^3/d (90 gpm). Shaft number 2 and 3 showed the highest potential for inflow with shaft number 6 showing the lowest potential for inflow.

After the completion of the modeling, Phase II of the dewatering program was considered; however, it was decided that the information gathered from Phase I was adequate for the contractors to estimate their dewatering requirements at each shaft. Therefore, the Phase II dewatering program was not implemented and the information gathered from the Plaxis flow model was used to develop the dewatering base line requirements in the contract specifications.

Geotechnical Instrumentation

The final aspect of design consisted of developing a plan for monitoring ground settlement and vibration which is critical to determine if the shaft or tunnel construction damaged nearby utilities, residential homes and other structures near the shaft and tunnel construction. Specifically, it was determined to use twenty three ground monitors and five facility monitors at key locations along the alignment to monitor potential for settlement of the ground and existing structures, respectively. Throughout the course of construction the tunneling contractor would manually survey these points to see if there was any differential settlement.

Table 4. Plaxis Flow Modeling Inflow Estimates

Shaft No.	Estimated Steady State Flow Range m^3/d (gpm)
1	81.8 – 190.8 (15 – 35)
2	109.0 – 436.1 (20 – 80)
3	81.8 – 490.6 (15 – 90)
4	27.3 – 327.1 (5 – 60)
5	27.3 – 136.3 (5 – 25)
6	27.3 – 81.4 (5 – 15)

During the course of construction, it was decided to have a third party prepare a separate monitoring plan to verify the tunneling contractor's geotechnical information. This was done due concerns from previous construction projects where damage occurred to residential homes and there was no independent monitoring of settlement or vibration of the monitors. In order to address this issue, an independent monitoring plan was developed which consisted of vibration sensors, settlement monitors and subsurface utility settlement monitors. These instruments were incorporated into the design at key locations on existing residential structures, buildings and utilities (including the existing 0.91 m (36-inch) PCCP force main). This program included automated vibration and settlement point monitoring which was used to retrieve the data from the sensors then uploaded to a project web site via a cell phone communication system. This would allow the design engineers to monitor the data for any significant changes on a daily basis without having to visit the site. This approach on the monitoring of the vibration and settlement was found to be more efficient in getting this information out to all interested stakeholders in real-time basis as opposed to manually collecting the readings. Figure 2 shows the GPS monitoring system that was used to upload the monitoring information to the website.

In addition to the settlement and vibration monitoring, a pre-construction survey was completed on specified homes which may be subject to vibration and settlement influence. This information along with the instrumentation data would help address potential damage claims to Fairfax County resulting from the shaft construction or tunneling operations.

The shaft and tunnel design along with the dewatering and geotechnical instrumentation requirements were used to finalize the contract documents which was completed in January 2011.

Figure 2. Solar Powered GPS Settlement & Vibration Monitoring System

CONSTRUCTION PHASE

In November of 2011 the tunneling contractor mobilized his crew and they began shaft 5 and 6 construction with the plan to launch the microtunneling boring machine (MTBM) from shaft 6 and have shaft 5 serve as the receiving shaft. On May 8, 2012 the contractor launched the MTBM from shaft 6. Figure 3 shows the MTBM being launched from shaft 6.

Figure 3. MTBM Launched from Shaft 6

On June 20, 2013 the MTBM reached shaft 5 which translates into a duration of 57 days of tunneling operations. The daily advancement of the MTBM between shafts 5 and 6 varied from a low of 3.05 m/d (10 ft./d) to a high of 21.3 m/d (70 ft./d) with an average of 10.7 m/d (35 ft./d). The advancement of the tunneling machine was dependent on several factors including subsurface conditions, equipment maintenance and other factors related to the tunneling machine. Generally, clay soil translated into lower advancement of the MTBM while sand or a more granular type soil allowed the MTBM to move forward at a higher rate. As noted there were several ground monitoring points along key locations to monitor settlement. GM-19 which was positioned near the Mount Vernon Highway next to shaft 5 started to show some signs of settlement occurring as the MTBM crossed under the road. The settlement was likely attributed to the sandy conditions near this location. This prompted the tunneling contractor to stop tunneling operations and inject chemical grout into the soil at defined locations to stabilize the subsurface conditions. Once the soil was stabilized the tunneling operations resumed and Figure 4 shows the MTBM being received at shaft 5 and concluding the first tunnel run for the Dogue Creek Force Main.

Figure 4. MTBM Received at Shaft 5

While the contractor was completing the tunneling run between shafts 5 and 6 they were constructing shafts 3 and 4. The design for shaft 3 was based on using sunk-in caissons construction due to the existing subsurface conditions which made it the best alternative for the shaft construction. However, the contractor proposed sheet piling as an alternative approach for this installation. It was decided to permit the contractor to use this alternative shaft design; however, they would be proceeding at their own risk to install this shaft and signed a waiver of claims for this method. The contractor began proceeding forward with the sheet pile design for the shaft and fairly quickly realized the sheet piling was not driving to the full design depth. As a result, they brought in a larger pile driving machine in an attempt to drive the sheeting to full design depth. This machine was unsuccessful in doing this so the tunneling contractor had to begin excavating out the shaft in lifts to reduce the frictional forces between the soil and the sheet piles. This process was used until the sheet piles were driven to full depth

The tunneling operation for all the shafts is summarized in Table 5. As shown all of the tunnel runs had a high degree of variability on the tunnel production rate with a high for all shafts of 21.3 m/tunneling shift (70 ft. /tunneling shift) to a low of 3.05

m/tunneling shift (10 ft./tunneling shift). The tunneling between shafts 3 and 4 as well as between 4 and 5 had overall a lower average production rate when compared to the other tunnel runs. Between these locations the tunneling contractor had a jacking frame hydraulic failure which caused delays in the work. Additionally, between shafts 4 and 5 the tunneling contractor had trouble steering the MTBM but was able to recover from the steering problems in the last 121.9 m (400 ft.) and was aligned on the target when it reach shaft 5.

Table 5. Tunneling Production Rates

Tunnel Run	Tunneling Length m (ft.)	MTBM Production Rate [m (ft.)/tunneling shift]		
		Maximum	Minimum	Average
1 to 2	265.5 (871)	21.3 (70)	3.05 (10)	10.7 (35)
2 to 3	237.1 (778)	21.3 (70)	3.05 (10)	9.75 (32)
3 to 4	352.7 (1157)	21.3 (70)	3.05 (10)	9.45 (31)
4 to 5	243.8 (800)	21.3 (70)	3.05 (10)	8.84 (29)
5 to 6	171.6 (563)	21.3 (70)	3.05 (10)	10.7 (35)

Slurry Dewatering

Another critical element in the tunneling operations is the need for dewatering the slurry from the tunneling operations. This operation essentially consists of a screening process to remove solids and larger particles then dewatering of the solids with a portable centrifuge. The centrate from the centrifuges is then recycled back to the tunneling operations to the head of the MTBM for lubrication of the cutting head. Figure 5 shows the typical dewatering operations used on this project.

Figure 5 –Tunneling Material Dewatering Operations

Shaft Dewatering

As noted in the design phase, Plaxis Flow modeling was performed to ascertain the anticipated ground water inflow for each shaft. The information from the model was then placed in the contract documents for bidding purposes. In reality, the dewatering requirements for each shaft were well below the model predictions. The tunneling contractor was able to control ground water flow with small submersible pumps located in a sump at each shaft. As a result, there were no major issues with keeping the shafts dewatered throughout the course of construction.

Geotechnical Instrumentation

Based on the results gathered from the vibration and settlement monitors, there were no issues related to vibration or settlement to nearby homes and buildings. Readings gathered from the sensors were all well below the thresholds which are defined in the contract documents. The maximum ground settlement reading recorded was 0.76 cm/sec (0.3 in/sec) which is well below the threshold or action limit of 1.27 cm/sec (0.5 inch/sec). Therefore, the construction of the shafts and tunnels did not impact any nearby structures or utilities.

SUMMARY AND CONCLUSIONS

Below is a summary of the key successes and lessons learned from the Dogue Creek Force Main project while implementing trenchless/tunneling technology:

- Visual and physical testing of the PCCP proved to be a valuable tool in assessing the condition of the existing force main and helped with the planning process.

- Trenchless/tunneling technologies are viable options for utilities that need to replace or install new force mains or pipe lines in environmental sensitive areas or where open trench technologies are not a viable means.

- Plaxis flow modeling is a useful tool in assessing dewatering needs for tunneling shafts; however, the model results should be validated with actual field testing if time permits.

- Microtunneling proved to be a good application of the technology for this project given the wetlands and the need to minimize obstruction with the Fort Belvoir Base operations.

- Production rates of the MTBM varied and are dependent on subsurface conditions; however, the average shift production rate for this project ranged between 8.8 m^3/d (29 ft/d) and 10.7 m^3/d (35 ft/d).

- The use of geotechnical instrumentation with automatic feedback and global positioning system to the design team proved to be useful in monitoring the ground settlement monitors from remote locations.

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Kaw WTP Water Transmission Main: Serving North Lawrence and Beyond

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Abstract
A preliminary hydraulic analysis and transmission main routing study was completed by Burns & McDonnell in 2008 to plan for projected growth in the southeast section of the City of Lawrence, KS (City) and to provide an additional water service feed to North Lawrence. The analysis recommended addition of a future 36-inch water transmission main to meet the City's needs. The nearly six mile water transmission main would connect to the Kaw water treatment plant (WTP), supply water to North Lawrence and ultimately provide service to the southeast section of the City to meet projected water demands beyond 2020. The City chose to design the first phase of the water transmission main project to connect the Kaw WTP to an existing 12-inch water main located in North Lawrence. The major challenge with serving North Lawrence is that the Kansas River separates this service area from the rest of the City. In addition to the Kansas River crossing, other design challenges associated with this project included a US Army Corp of Engineers levee crossing, two railroad crossings, a creek crossing, and work in the Burcham Park area which is a high use park enjoyed by the residents of Lawrence. Fusible polyvinylchloride pipe (FPVCP) was selected and bid for the transmission main. At the time of installation, the 2,400 foot Kansas River crossing was the longest FPVCP horizontal direction drilling (HDD) installation in the world for 36-inch diameter AWWA C905 Dimension Ratio (DR) 21 pipe, which is currently the highest rated pressure class available in 36-inch FPVCP.

Background
The City of Lawrence, Kansas is located approximately 25 miles west of the Kansas City metropolitan area. Lawrence has a population around 91,000, and it is home to the University of Kansas, which has over 20,000 students at the Lawrence campus. The City currently treats and provides water to customers within the City and provides five (5) Rural Water Districts, the University of Kansas, and the City of Baldwin, Kansas with wholesale treated water. Two of the wholesale water customers have a long term interest in receiving additional water. One of those wholesale water customers provides water to two additional municipalities. Both of the connections with these wholesale customers are located in the southeast area of Lawrence.

In addition to supplying potable water to an increasing customer base, the City recognized that having a redundant water source to the North Lawrence area would increase water supply reliability. As shown in Figure 1 below, the area of North Lawrence is separated from the rest of Lawrence by the Kansas River. At the time of the study, the City supplied potable water to North Lawrence through a single 16-inch transmission main crossing the Kansas River. This river crossing is an aerial crossing connected to the Highway 59 Bridge. In recent years, sections of pipe in this aerial crossing have developed pin-hole leaks causing ongoing maintenance issues and supply reliability concerns. The deteriorated pipe sections were repaired but this condition brought the need for an additional water feed for the North Lawrence area to the forefront for City water supply planners.

Figure 1 – City of Lawrence, Kansas

Based upon routing study recommendations, the City decided to phase the construction of a new water transmission main, with the first phase including the Kansas River crossing to provide a redundant feed to North Lawrence. Subsequent phases will ultimately extend the transmission main to the southeast area of Lawrence. When complete, the nearly six mile water transmission main will connect to the Kaw WTP, supply water to North Lawrence, and ultimately provide service to the southeast section of the City to meet projected water demands beyond 2020. The City selected Burns & McDonnell to design the first phase of the water transmission main project. This phase of the project included approximately 1.25 miles of water transmission main connecting the Kaw WTP to an existing 12-inch water main located in North Lawrence.

Design Criteria/Challenges
A hydraulic analysis was performed to determine the required transmission main size. The modeling took into account the potential high growth development possible in the southeast area of Lawrence. It also considered the requested future water demands from their existing wholesale customers beyond 2020. The hydraulic analysis indicated the transmission main, from the connection at the Kaw WTP to the final connection in the southeast area of Lawrence, should be 36-inch diameter.

With the size determined for the entire transmission main route, an analysis was performed to determine the necessary pressure rating for the first phase of the project. The main factors considered to assess the required pressure rating were the high service pump performance curves, potential surges that may be generated within the system, and the potential elevation changes throughout the first phase of the transmission main. Analysis results recommended a pressure rating of at least 200 pounds per square inch (psi).

Another factor in the design of the transmission main was the potential for high corrosive soils in the area. The City has experienced the effects of high corrosive soils on their water mains in the past. Preliminary design work included a geotechnical investigation of the project area. The resulting investigation report indicated at least one area where the soil was considered corrosive. In addition, it was noted in various locations that groundwater was present at the proposed trench depths. Based upon reported findings and the limited accessibility of the pipe, once installed, a cathodic protection system would be required if a metallic pipe material were used.

The transmission main alignment also included numerous challenges. Figure 2 shows the proposed transmission main alignment. The alignment crosses the Kansas River into the North Lawrence area. The Kansas River in the area of the transmission main crossing has a US Army Corps of Engineers (USACE) levee system in place to protect the North Lawrence area from flooding with river water. The river and levee crossings both required authorization and permits from the USACE. The authorization and permitting process required conformity to established USACE design criteria.

The alignment also crosses two existing railroad right-of-ways. The Burlington-Northern-Santa Fe (BNSF) Railroad crossing is located near the connection point to the Kaw WTP. The Union Pacific (UP) Railroad crossing is located almost adjacent to the levee on the east side of the river in North Lawrence. Both of these crossings required permits from each railroad. Each railroad company required water line crossings to be installed within a steel casing pipe in compliance with the American Railway Engineering and Maintenance-of-Way Association (AREMA) standards. AREMA Standards require the steel casing extend across the entire width of the railroad right-of-way.

As shown in Figure 2 below, the water transmission main alignment runs through Burcham Park. Burcham Park is a highly used park that contains walking trails, playground equipment, and access to the river. The park also includes a boathouse, which is home to the University of Kansas rowing team. The City required that access to the park and boathouse be maintained throughout construction.

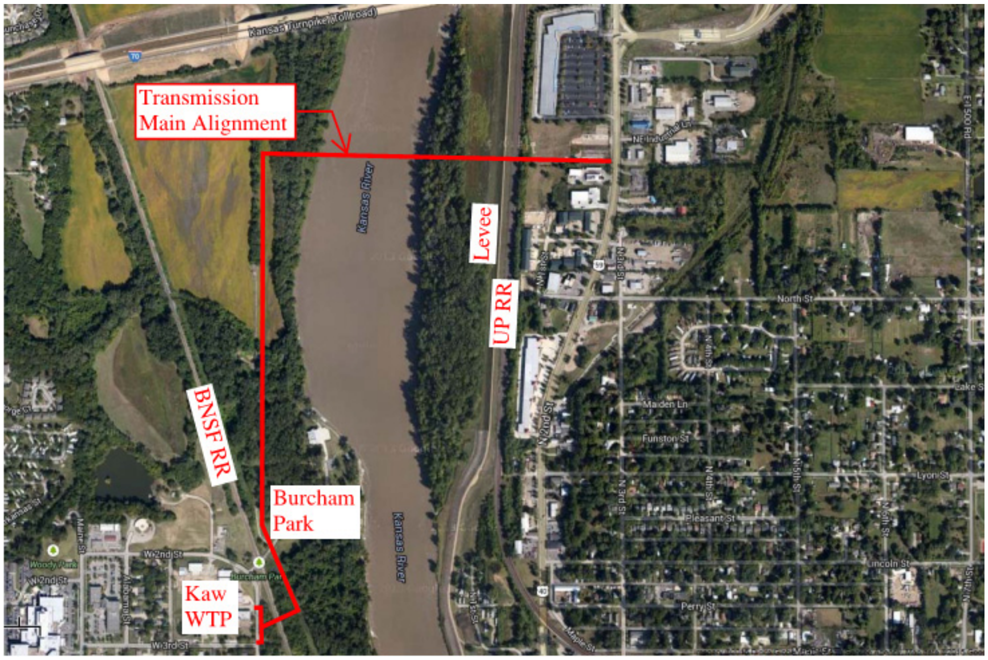

Figure 2 –Water Transmission Main Alignment

The park grounds and surrounding areas contain numerous trees that the City requested not be disturbed. In addition, there are two wetland areas and a stream crossing. Construction activities in these areas had to be minimized or accomplished without significant permanent impact to the wetlands or stream. Because of all these items, it was necessary to limit the construction zones throughout the project and investigate alternatives to open-cut installation construction methods.

Pipe Material Evaluation

Prior to finalizing the methods of construction, it was necessary to determine what pipe material would be utilized. As part of the design process, a pipe material evaluation was completed. Table 1 lists the various criteria considered in the evaluation, and Table 2 lists the pipe materials evaluated.

Table 1 – Pipe Material Evaluation Considerations

Design life
Corrosivity resistance
Availability in 36-inch diameter
Feasibility of installation
Availability of required pressure rating

Table 2 – Pipe Materials Evaluated

Steel pipe
Ductile iron pipe (DIP)
High-density polyethylene (HDPE)
Fusible polyvinylchloride pipe (FPVCP)

The potential design life of each of the pipe materials evaluated is reported to be in the range of 100 years. The actual useful life of the transmission main will depend upon a number of factors including the site conditions, the quality of installation, the pressure ratings and associated wall thicknesses, and installation depths.

Each of the pipe materials is available in 36-inch diameter and at the 200 psi pressure rating required. It should be noted that when the initial pipe material evaluation was performed, 200 psi (DR 21) FPVCP was not yet available from the manufacturer. FPVCP became available in 36-inch diameter with a DR 21 pressure rating during the time between the study/preliminary design phase and completion of the final design.

The two main deciding factors in the pipe material evaluation were the corrosivity resistance of the material and the feasibility of construction. The four pipe materials evaluated can be divided into two main categories: metallic and non-metallic. Because the City has experienced corrosion issues on some of their existing water mains, and because the geotechnical evaluation identified an area of corrosive soil along the proposed alignment, use of metallic pipes would require installation of a cathodic protection system. As an additional layer of corrosion protection, specialized coatings on the metallic pipe may also have been required.

The corrosion resistance of the non-metallic pipe materials under consideration was well documented. Consequently, a cathodic protection system would not be required if non-metallic pipe material is selected. However, if metallic fittings were utilized with the non-metallic pipe system, then these fittings would require corrosion protection using polyethylene encasement.

The installation feasibility evaluation for the various pipe materials took into consideration that the river crossing would most likely be installed using horizontal directional drilling (HDD) installation methods. Because the pipe is below the river future maintenance of the pipeline was also a concern. Both these considerations led to the conclusion that a joint-less system should be utilized for this crossing to facilitate HDD installation and minimize future maintenance needs. Depending upon the pipe thickness required the bore hole for joint-less pipe is typically smaller than a ball and socket or restrained joint bell and spigot type pipe system. A joint-less piping system will not be subject to future failures associated with deteriorated gaskets or loose joints. The installation feasibility assessment eliminated DIP from further consideration for use at the river crossing portion of the alignment.

The installation feasibility analysis evaluated the suitability for using HDD methods to install HDPE pipe for the river crossing portion of the alignment. Design guidelines found in ASTM F1962-05 were used to evaluate suitability of DR 11 HDPE. The analysis of critical buckling pressure during pullback resulted in a factor of safety against buckling during pullback that was below the standard minimum recommended value. The analysis results eliminated HDPE pipe from consideration for installation at the river crossing.

The installation feasibility analysis evaluated the suitability for using HDD methods to install FPVCP for the river crossing. Design guidelines found in ASTM F1962-05 were used in conjunction with guidelines in ASCE MOP 108 to evaluate suitability of DR 21 FPVCP. Installation forces and long-term operational loads were considered. Pipe deflection, critical buckling pressure, and anticipated and allowable tensile stress were evaluated. The analysis results indicated FPVCP was an acceptable pipe material for installation at the river crossing.

At this stage of the pipe material evaluation, the City expressed a preference for using the same pipe material throughout the project. FPVCP and steel pipe materials remained for further consideration. Corrosivity concerns and the resulting corrosion protection systems that would be required to be installed and maintained with steel pipe were considered. A major concern for the steel pipe was how the coating system on the outside of the pipe would hold up during the HDD installation process since much of the HDD installation was through solid rock. This concern eliminated steel pipe from further consideration. FPVC met all the necessary requirements for the project and was selected and bid for the transmission main installation project.

Design and Installation Methods

Several installation methods were evaluated for use to construct this project. The methods considered included open cut, jack & bore utilizing a casing pipe, and HDD. The selected method depended upon requirements for each of the specific project areas along the alignment. Open cut installation methods were originally proposed for use wherever feasible. The typical installation for a pipeline across an USACE levee is to install it using open cut methods over the top of the levee and then repairing the levee after installation. Railroads typically require installation by jack & bore, so this was the original plan. Then HDD would be used for installation under the river; however, once design progressed, these initial selections were revised.

As shown in Figure 2, the transmission main alignment crosses two railroad right-of-ways. The railroad requirements called for the transmission main within the railroad right-of-way to be installed by a jack & bore method. This required a bore pit to be excavated. The boring machine and casing pipe were placed within the boring pit. The casing pipe was then “jacked” into place by hydraulic jacks while a cutting head on a rotating helical auger was used to remove the spoils from within the casing pipe. Once the casing pipe was in place, the carrier pipe was installed. The railroads typically require the steel casing pipe to be installed the entire width of the railroad right-of-way. This installation method worked for the BNSF railroad crossing near the Kaw WTP, but not the UP railroad crossing on the east side of the river near the levee.

As shown in Figure 3 below, the levee and UP railroad are adjacent to each other. The 30-feet between the levee centerline and the west edge of the UP right-of-way provided insufficient space for a bore pit. It would have been possible to excavate the bore pit on the east side of the railroad right-of-way, but working between the levee and the railroad would have been very confined, so an alternative installation method was considered.

Figure 3 – East Side of the River – Levee/Railroad Crossing

As noted in the Pipe Material Evaluation section, ASTM F1962-05 and ASCE MOP 108 were used as a basis of design for the river crossing. Due to the need to cross the levee and railroad, a radius of curvature was used for the layout and design that was greater than the minimum allowable bend radius for FPVC. This led to a longer bore profile curve which reduced the pull stresses during installation. The actual radius of curvature for the bore was 3600-feet with the minimum allowable bend radius for 36-inch FPVCP of 798-feet. Data obtained from nearby soil borings indicated a layer of solid rock existed below the river. The HDD was designed for the bore profile to be within this layer of rock to reduce the risk of the bore hole collapsing. Given the location of the rock layer, the radius of curvature of the proposed transmission main, and the limits on the allowable entry and exit bore angles for a bore this size, it was not feasible to bring the bore up in the area between the river and the levee.

Exceptions were negotiated for the open cut levee crossing requirement and the railroad casing requirement. The USACE does allow levees to be directional drilled if certain requirements are met. One requirement indicates if soil borings of the project site are provided, then the directional drill must penetrate the substratum a minimum of 300-feet from the levee centerline on the landside and may not exit closer than 300 feet on the riverside. For a conservative approach, the bore profile was designed to maintain a 500-foot offset from the directional drill penetration to the centerline of the levee, as shown in Figure 4 below. It was also designed for the bore to be in the rock layer under the levee crossing, which is approximately 70-feet below the top of the levee.

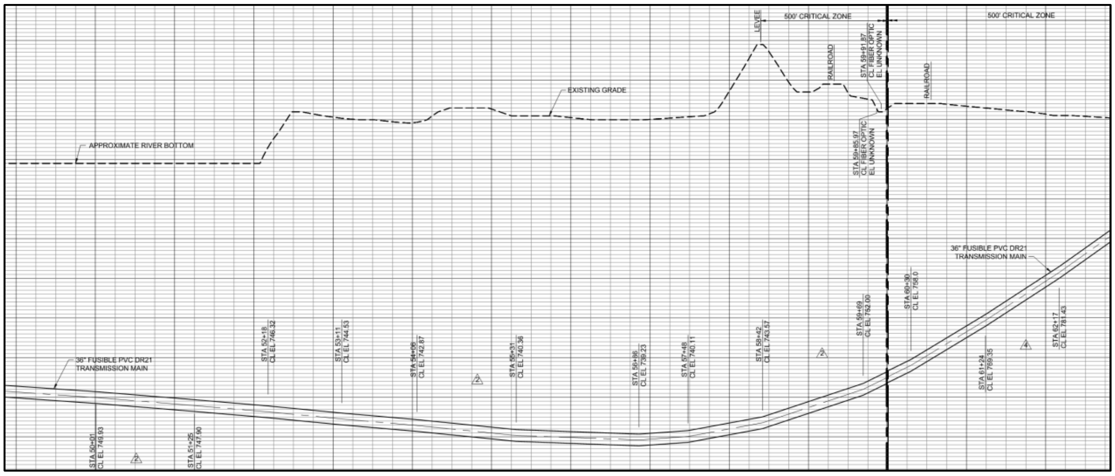

Figure 4 – Kansas River Bore Profile

With the proposed bore approved by the USACE, an exception was needed to the casing pipe requirement from the railroad. Through the use of the geotechnical reports and showing that the bore was designed to be within the rock layer as it crossed underneath the railroad, the railroad granted an exception. This exception was granted because the rock layer in effect is acting as a solid casing pipe below the railroad, and the crossing is nearly 70-feet beneath the railroad so the potential for settlement is low.

With the river crossing and railroad crossing set, the area in and around Burcham Park offered the remaining installation concerns. As previously mentioned, the City required access to the park and boathouse to remain open at all times, and tree removal must be at a minimum. The other concern with the crossing in the park was the number of utilities existing in the area. The City has raw water wells and a river intake in the area along with water distribution lines and other utility lines. Existing utility crossings required the transmission main to be at least 12-feet deep in this area. An open excavation of this depth near the park entrance was not feasible. To avoid this, and to minimize tree removal within the park area to the north, HDD installation was designed for this area. There is also a stream running east-west along the north side of the park property, and an open cut stream crossing was avoided by utilizing HDD through this section.

Construction Phase
The Kansas River crossing was one of the high risk installation sections on the project. The River bore was the first bore completed on the project. The river crossing measured approximately 2,400 linear feet. The pipe reaming process for this HDD bore took nearly 56 days. The pilot hole exited the ground approximately 10-feet east and 3-feet north of the design point.

The drilling contractor began the HDD process by starting with a 10-inch diameter pilot hole. Once the pilot hole was through, a 26-inch diameter back ream was used. When the 26-inch back ream was complete, the drilling contractor went to a 38-inch back ream, but due to low productivity switched to a 32-inch and then went back with the 38-inch. The next back ream measured 42-inches in diameter, and the final back ream was 48-inch.

As shown in Figure 2, there was limited laydown area for the pipe string. The west side of the property is bound by the BNSF railroad, and the north side is bound by the I-70 Interstate. Because of this, the required 2,400 foot river crossing was initially fused in three lengths of 1,100 feet, 880 feet, and 420 feet, which required two intermediate fusions to occur during the pullback process. With the 48-inch bore hole open, the pipe pullback process began. Initial pull back activities began at 6:00 in the morning, with the first section of pipe being pulled through about 9:00 in the morning. Each intermediate fusion processes took approximately two hours. After 16.5 hours, the last section of pipe was successfully pulled into place at 1:30 the following morning.

Figure 5 – Kansas River Bore Pipe String

After the Kansas River crossing was complete, subsequent HDD sections through Burcham Park were also installed without issue. Construction of the entire first phase of the transmission main was completed in January 2015, and water is currently being supplied to North Lawrence through this pipeline.

Conclusion

The City of Lawrence planned ahead and recognized the need for a redundant water feed to North Lawrence and additional water supply to the southeast of the City to meet future water demands. The first phase of this transmission main project provided the redundant water feed to North Lawrence.

Various pipe materials were considered and evaluated for design life, corrosivity resistance, size and pressure rating availability, and feasibility of installation. Ultimately, FPVCP was selected and bid for the transmission main.

The project included numerous design and permitting challenges including the Kansas River crossing, a US Army Corp of Engineers levee crossing, two railroad crossings, a creek crossing, and crossing the Burcham Park area. Many of these challenges were overcome in part by designing and utilizing a HDD installation method instead of the standard open cut method.

Construction of the first phase of the transmission main was completed in January of this year. The transmission main has a connection point available for the next phase of the project and water is currently being supplied to North Lawrence through this pipeline. At the time of installation, the Kansas River crossing was the longest FPVCP HDD installation in the world for 36” diameter AWWA C905 DR21 pipe, which is currently the highest rated pressure class available in 36” FPVCP.

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Permitting Requirements Drive Trenchless Design and Project Risk: An HDD Pressure Pipeline Case History

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Abstract

The Denver Metro Wastewater Reclamation District (MWRD) is installing a 10-inch pressure pipeline to carry potable water from the Robert W. Hite Treatment Facility located in Denver, Colorado to the MWRD facility. The pipeline alignment crosses beneath both the Burlington Ditch, owned and operated by the Farmers Reservoir and Irrigation Company (FRICO), and the Union Pacific Railroad (UPRR) trestle, both of which have specific and stringent requirements for trenchless pipeline installations. Furthermore, the pipeline traverses Denver Water right-of-way, thus triggering the necessity to adhere to another set of trenchless installation requirements. This paper presents challenges encountered during design and construction of the horizontal directional drill, focusing on how the design and construction was influenced by FRICO, UPRR, and Denver Water requirements. Discussion of the settlement and hydrofracture analysis performed during design is presented and compared to what was observed in the field during construction. Furthermore, this case study details the morass of permitting requirements encountered and how, as a whole, the requirements actually dictated the overall risk profile of the project.

INTRODUCTION

The Denver Metro Wastewater Reclamation District (MWRD) is installing a 10-inch pressure pipeline to carry potable water from the Robert W. Hite Treatment Facility located in Denver, Colorado, to the MWRD facility. The 10-inch pipeline will connect to an existing Denver Water 24-inch main and traverse the Denver Water property within the Robert W. Hite Treatment Facility before passing beneath the Union Pacific Railroad (UPRR) Bridge and Burlington Ditch, owned and operated by the Farmers Reservoir and Irrigation Company (FRICO). The water pipeline, 1,800 feet in length, is designed primarily with traditional open cut installation except for a 700-foot horizontal directional drill (HDD) crossing of the UPRR and Burlington

Ditch. Figure 1 shows the HDD alignment and profile including geotechnical conditions.

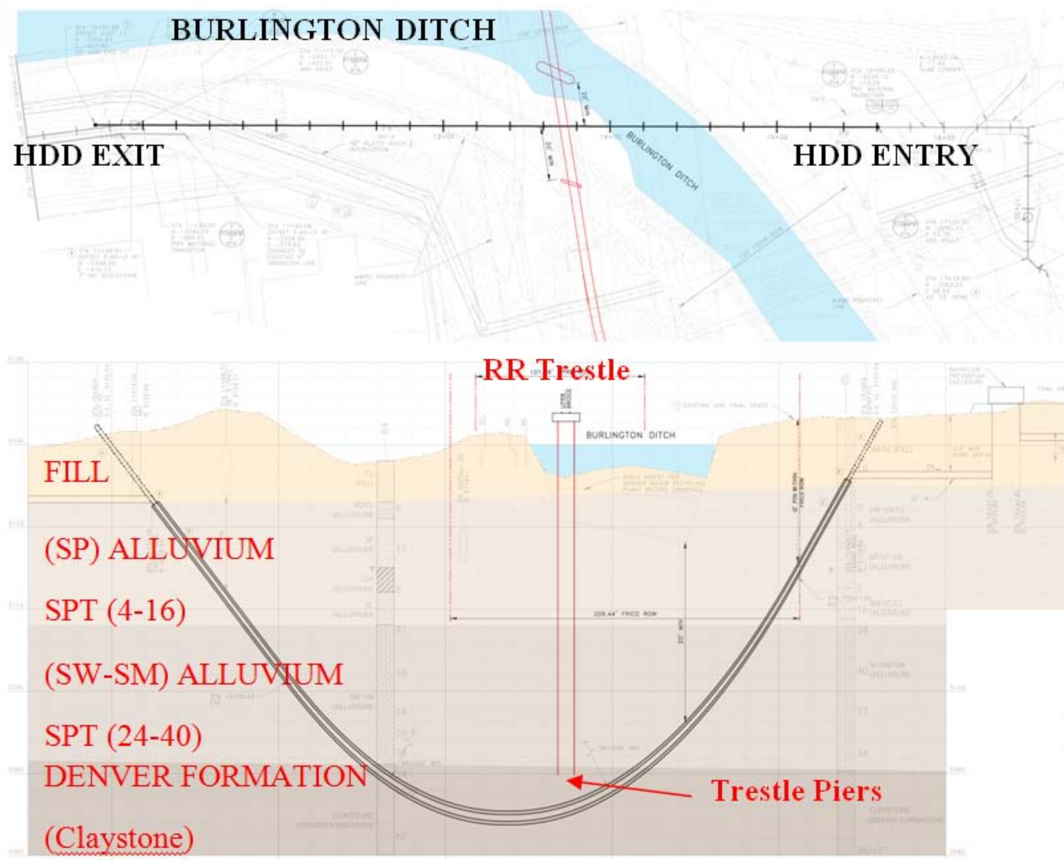

Figure 1. Geotechnical Profile along Preliminary HDD Alignment.

Denver Water, UPRR, and FRICO each had independent trenchless construction standards and requirements that had to be accommodated in the HDD design to obtain the necessary permits for construction. The major challenge of the HDD design was not the requirements of any one agency, but the conglomeration of requirements from all three. The initial approach was to offer a thoroughly studied HDD design with the least possible degree of constructability risk, requesting variances from the interested permitting entities where the design deviated from particular standards. This approach was roundly denied by UPRR and ultimately led to a complete re-design of the HDD, which was based primarily upon selecting the most arduous requirements of the three permitting agency standards and requirements regardless of the impact to overall project risk. The resulting increase in risk was then mitigated through additional specification requirements. This paper describes the initial design approach as well as the final design, incorporating accommodations for requirements of the three permitting agencies. The construction activities are also summarized and the influence of the permit regulations and risk mitigation strategies developed during design on the overall success of the project are discussed.

GEOTECHNICAL CONDITIONS

Soils along the HDD alignment generally consist of fill overlying alluvium, which in turn overlies sedimentary bedrock of the Denver Formation (Shannon and Wilson, 2013). The fill soils along the HDD alignment range from 3 to 11 feet below ground surface and consist of medium stiff silty to sandy clay. The upper portion of the underlying alluvium is roughly 15 feet thick and generally consists of loose to medium dense, trace to silty or clayey, gravelly sand with scattered cobbles. Soil types in the lower alluvium are more uniform at approximate elevation 5,108 feet (or approximately 25 feet below ground surface), consisting of medium dense to dense, slightly silty to silty sand with trace gravel. Sedimentary bedrock of the Denver Formation was encountered approximately 40 feet below ground surface, or elevation 5,090 feet. The bedrock typically consists of very low strength (< 700 psi unconfined compressive strength), fresh to slightly weathered claystone, siltstone, and sandstone (Shannon & Wilson, 2013). Figure 1 illustrates the geotechnical conditions that were anticipated along the preliminary HDD alignment.

INITIAL DESIGN APPROACH

The HDD design submitted for the initial permit applications (initial HDD design) focused on reducing overall project risk through a balanced approach that addressed trenchless construction risks, encapsulating those associated with drilling near adjacent facilities and other constructability risks. The initial design only incorporated those elements of the three agency standards and permit restrictions that fit within the initial design philosophy of reducing trenchless risk. Thus, requests for variances from the permitting agency standards were submitted for requirements that posed an increased risk to the project. A primary variance from the standards was the use of a plastic casing to encapsulate the 10-inch fusible polyvinyl chloride (FPVC) carrier pipeline, instead of a steel casing pipe. The use of a plastic casing with a pressure capacity of one and a half times that of the carrier pipe allowed for deepening of the HDD bore path within the FRICO and UPRR properties, due to the smaller bend radius of plastic pipe versus steel pipe. As a result, the bore path would traverse beneath the railroad trestle piers approximately 50 feet below the ground surface. This was desirable, since the UPRR bridge as-built information could not be obtained and the depth of the bridge piers were unknown. It was reasonable to assume the trestle piers extended to the Denver Formation to attain a high tip bearing capacity. All design analyses and calculations were based upon this assumption.

Figure 2 shows the analysis of the UPRR trestle piers, the results of which were then compared to the potential influence zone of the HDD bore. The results of the analysis demonstrated that there was no overlap of the soils supporting the bridge and the potential zone of disturbance caused by the HDD installation. Parameters of the initial bore geometry were adjusted to ensure, as much as reasonably possible, that the HDD would not impact the UPRR trestle, the FRICO irrigation ditch, or any of the crossing utilities.

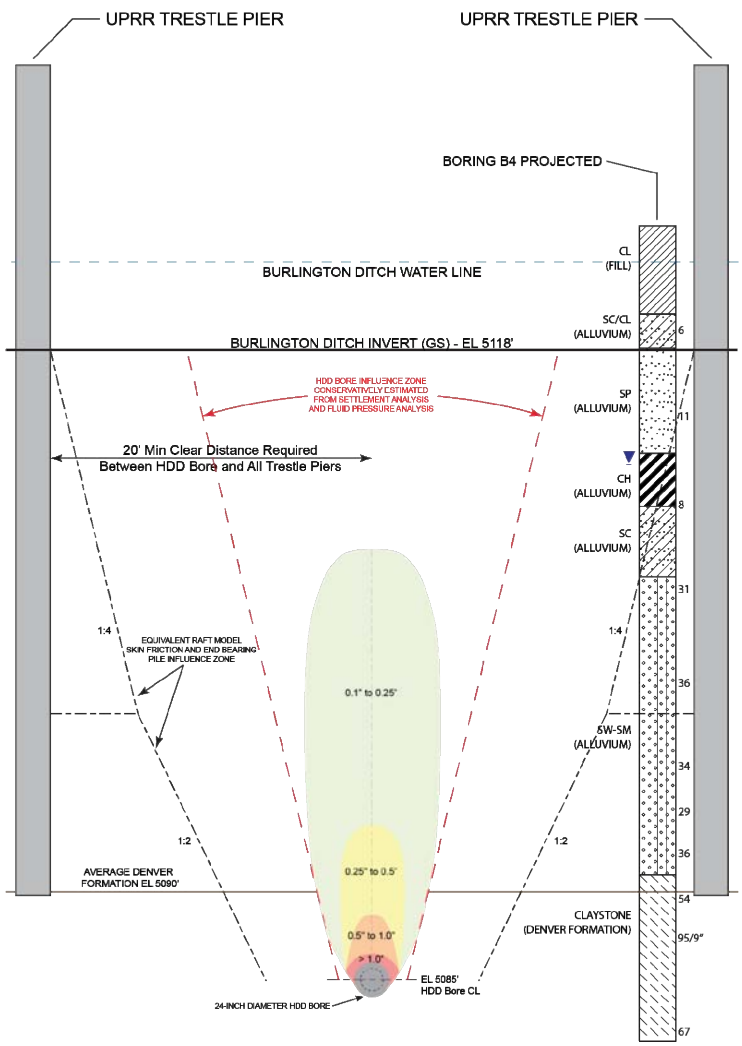

Figure 2. Analysis Performed to Determine the Influence Zone of the HDD Bore and UPRR Trestle Piles.

The anticipated geotechnical conditions were considered in the initial design of the HDD bore geometry. Given their density and fines content, the near-surface alluvial soils were considered relatively good for directional drilling. On the other hand, these soils contained cobbles, which can pose borehole stability problems and steering issues. Improved drilling was expected with depth, within the deeper soils absent of scattered cobbles. The initial design accounted for this with a straight trajectory from the entry that would not require steering until the lower, more stable, uniform soils were encountered. A steep entry angle of 20 degrees was selected to reach the more competent alluvium quickly and gain depth while within the FRICO ROW.

The bore was initially designed to be entirely within the more competent soils while within the FRICO ROW, and enter into the sedimentary bedrock of the Denver Formation while passing between the UPRR bridge piers (Figure 1). Thus, the bore geometry was chosen to minimize the distance traversing soft or loose soils, maintain the bore in the ideal competent soils while in the FRICO ROW, and ensure the bore was fully within bedrock when passing between the UPRR bridge piers. The use of a plastic casing pipe allowed the flexibility necessary to adjust the borehole geometry parameters in response to the anticipated geotechnical conditions. An additional benefit to using a plastic carrier pipe was the lack of necessity to cathodically protect it to prevent corrosion. A steel casing, on the other hand, would likely require cathodic protection or an increased wall thickness to provide an effective service life of equal duration.

In addition to the request to utilize plastic instead of steel casing, the initial design eliminated the use of casing spacers between the 10-inch carrier pipe and the 16-inch casing pipe. By doing so, the casing pipe and HDD borehole diameters could be reduced. Decreasing the borehole diameter lowered the potential for surface settlement near the entry and exit locations. Furthermore, the smaller diameter proportionally increased the separation distance of the bore path from the bottom of the FRICO irrigation ditch as well as the existing utilities to be crossed.

HDD RE-DESIGN FOR THE MOST ARDUOUS PERMIT REQUIREMENTS

Despite the careful analysis and technical documentation presented to explain the reduced risks that could be realized by variances from permitting standards, the variance requests and permit applications were denied. After the initial design and variance requests were rejected, a second design was pursued based on an entirely different philosophy. The re-design was a study in mitigating extremes and consisted of selecting the “worst case” parameter for each of the permit requirements. At over $20,000 per application, it is important that the HDD designer be able to understand and abide by the imposed permit requirements for HDD designed crossings. Currently, the myriad of conflicting, often inapplicable, requirements make this a difficult proposition at best.

A table combining the permitting requirements of the three interested agencies was developed to track the selection of the various design parameters along with mitigation efforts added to offset resulting increases to project risk. Table 1 shows a condensed version of the spreadsheet that was used to select the dominant, i.e., most arduous, requirements and the proposed mitigation efforts. Approximately one-third of the requirements, along with the requirement references, have been omitted from Table 1 for clarity in publication.

It is clear when examining current standards developed for the various agencies that requirements have been developed as “catchall” standards that are improperly employed for all trenchless installations. The more dated requirements are not remotely applicable to HDD installations. For instance, the 2013 AREMA Manual for Railway Engineering requires that the “maximum borehole diameter will be no more than 2 inches larger than the outside diameter of the installed carrier or casing pipe” (AREMA, 2013). This requirement is in direct contradiction to the HDD industry accepted practice of sizing the final borehole diameter to be either 12 inches larger or 1.5 times the outside diameter of the installed pipe, whichever is less (Bennett, D., and Ariaratnam, S., 2008). Presumably, the 2-inch maximum overcut requirement was originally developed for pipe jacking installation methods, where it is applicable to prevent excessive systematic settlement within the ROW. Typically, HDD installations are much deeper than pipe jacking installations, thus reducing the likelihood of potential surface settlement. It is the author's opinion that in lieu of a 2-inch overcut restriction for HDD, the agency would be better served by eliminating the oversized steel casing and casing spacer requirements for HDD crossings. This would actually reduce the final borehole diameter and dramatically decrease the magnitude of potential ground disturbance during construction and subsequent potential long term ground surface settlement.

Table 1. Comparison of Requirements Incorporated into the Final Design. The "Most Arduous" Requirements are Highlighted.

AREMA Requirements	FRICO Requirements	Denver Water Requirements (Material Spec - 34)	Design Response:
Pipelines must be encased in larger casing	Steel casing must be full width of FRICO ditch	Welded steel must meet AWWA C200	Design will comply accordingly
Cross tracks at 90 degrees and no less than at 45 degrees			End points will control… will be close to perpendicular to RR
Pipeline must be able to be electronically located	Marker posts required		Locate wire will be pulled with pipeline
Carrier pipe joints shall be leak proof or welded	Carrier pipe shall be restrained joints		fPVC will be butt fused for entire length
PVC and PE pipe are approved carrier pipe materials		Carrier </=20", Certa-Lok RJ or fPVC pipe.	fPVC carrier pipe will be used
MAOP = 100 psi, Plastic carrier pipe conform to ANSI B31.3	Pipe pressure rating 50% greater than outside ROW Pressure test is required		Design requires a MOAP of ~200 psi
Casing ID 4" > than carrier pipe joint OD	Min. 3" clearance around carrier pipe	Casing 10" larger than carrier	DW for jacking Will use 16" casing
Steel casing SMYS of at least 35 ksi		Minimum yield strength of 35,000 psi	Design will comply accordingly
Casing pipe shall extend a min. 25 ft from outside track when casing is below ground			Design will require several hundred feet of setback
Min casing wall thickness (not coated or catholically protected) 12.75" -- 0.188" / 14" -- 0.250" / 16" -- 0.281" / 18" -- 0.312"	Casing >12" but <24" diameter = 0.25" wall steel pipe Casing shall be suitably protected from failure due to corrosion for a design life of 50 years.	Designed to withstand applied loads External loading shall be AASHTO H20 HWY or RR loading plus jacking loading, E-80 RR loading Min. wall = 0.375"	Design will use the most conservative wall thickness requirement of 0.375" to comply
Casing installed to prevent the formation of a waterway under the railway	Casing shall be liquid tight & casing sealed to the carrier pipe at each end	EPDM or neoprene rubber end seals on casing	Design will use casing end seals per Denver Metro requirements
	Insulated casing spacers required	Casing spacers required	Design will utilize slim casing spacers
Casing pipe not less than 5.5 ft from base of rail to top of casing	Top of the proposed pipe shall be not less than 12 ft below the canal invert		Hydrofracture design will control Min. depth of 20 ft below ditch
3 ft min. cover at shallowest point	3 ft min. cover within FRICO ROW		Hydrofracture design will control Min. depth of 12 ft within ROW
The location of the bore must not conflict with any facilities within the RR ROW			Cannot pothole utilities within ROW until permits in construction
Design track bores to be >150 ft from the nearest bridge, culvert, road crossing, signal structure, track switch, building or other major structure			Permit application and design will conservatively treat going between the bridge piers as if it were a standard RR embankment
Design bore pits to be a min. of 30 ft from centerline of track when measured at right angles to the track	Case bore operations under FRICO canals shall be conducted outside the FRICO ROW and the bore pits shall also be located outside the FRICO ROW		Design is extremely conservative in this regard and will fully comply
Max borehole diameter no more than 2" larger than the OD of the casing pipe			Not possible with HDD construction method Good Practices
Min depth of 5 ft under natural ground, or 12 ft under base of rail			Hydrofracture will control and exceed the depth requirement
Constant slope for min. 30 ft from CL of track, 2 ft beyond toe of slope, and 3 ft beyond ditch, whichever is greater			Will provide a 60 ft tangent section (0% slope) of the bore within the UPRR ROW to meet this criteria

The permitting standard that resulted in the greatest impact to the re-design was the requirement of steel as the casing material. Additionally, the requirement that casing spacers be used to place the carrier pipe within the steel casing with a minimum radial annulus of 3 inches severely limited adjustments to the bore geometry. The use of 16-inch steel casing necessitated a bend radius of 1,600 feet, over three times larger than the 500-foot bend radius required for the plastic casing pipe. This ultimately reduced the amount of separation below the FRICO ditch by 8 feet, as well as reducing the total depth of cover below grade by 18 feet where the bore passed between the UPRR bridge trestle piers. A balance was sought between achieving an appropriate depth of cover below the critical crossing locations to prevent hydrofracture and ensuring entry and exit tangents that would reduce the risk of having steering difficulties in the less competent near-surface soils.

A much shallower exit angle was also incorporated in the re-design to reduce the difficulties of lofting the steel pipe during pullback. Pipe layout was also problematic when acquiring sufficient space to allow the contractor the ability to weld the entire length of the casing pipe above ground prior to pullback. There was limited available layout space in the Denver Water property that could be used by the contractor; however, development of arduous specification restrictions allowed the contractor to use a delineated portion of the property provided minimal disturbance occurred.

CONSTRUCTION

The re-designed bore path crosses numerous existing utilities, many of which could not be potholed until start of construction (Figure 3). The 78-inch Platte River Interceptor pipeline is crossed in two locations, once at entry and again near the exit point. Other nearby utilities include a duct bank, large fiber optic cable, and two diesel fuel lines within the UPRR ROW. Key utility potholes, including the two diesel fuel lines and large fiber optic communication line, were only allowed to be potholed during construction after the contractor obtained the necessary permits. During design, it was assumed that the fuel lines were located at an approximate depth of 8 feet below grade based on available information from the utility owner; however, the lines were actually found to be located at an average depth of 23 feet below the ground surface.

Locating the fiber optic line proved to be quite difficult and became extremely time consuming and costly. After several weeks of attempting to advance potholes, which were vacuum excavated and cased to depths sufficiently below the elevation of the HDD bore on both sides of the bore centerline, the decision was made to proceed with the HDD bore without actually locating the line. The owner of the fiber line suggested that the line was 10 or more feet below the termination depth of the exploratory potholes, although they did not actually confirm the depth of the line. After discussions with the contractor, it was decided that the HDD would commence and that as long as the contractor did not deviate from the specified vertical tolerance of ±3 feet, it would be unlikely to impact the un-located fiber optic line.

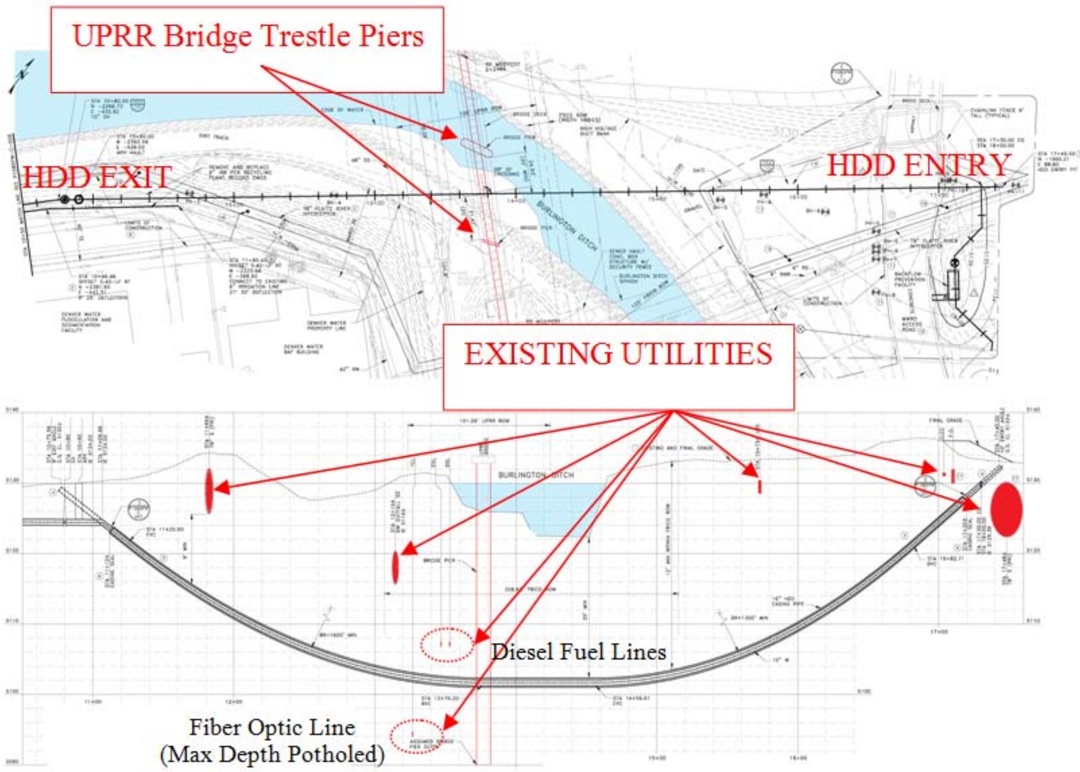

Figure 3. Final Bore Path with Obstacles Including Existing Utilities.

In an attempt to help the contractor expedite submittals, the construction management team agreed to allow piecemeal or partial submittals to be submitted in whatever order or level of completion the contractor deemed appropriate. This submittal process proved to be less than ideal, and in response, an extensive tracking spreadsheet was developed to track submittals by level of completion along with detailed comments for incomplete or conflicting submittals. The degree of difficulty for submittal preparation was certainly impacted by the morass of requirements included in the HDD specification necessary to ensure compliance with the three permitting agency standards and guidelines. Ultimately a series of conference calls, including regular distribution of the submittal tracking spreadsheet to the contractor proved successful to increase the efficiency of the submittal process. The construction management team facilitated regular discussions between the trenchless subconsultant and the trenchless subcontractor during the submittal review that not only expedited the process, but also developed a line of communication that proved invaluable when issues occurred during construction.

One of the primary issues that presented itself early on in the project was the area available for setup of the drill rig at the proposed entry point. The contractor decided to mobilize a larger drill rig which would allow forward reaming with a single upsize to the final 24-inch bore diameter. However, the use of a rig larger than anticipated during design necessitated a change in the entry location and partial removal of an existing high security fence. The design team quickly incorporated the new entry point, bore path changes, and pothole utility information into a revised HDD bore path that allowed the contractor space for equipment while complying with the various permit restrictions. During this phase of the project, the contractor also proposed employing a 4-inch drilling fluid return line with a mud pit pump setup at the exit pit which was fused together beneath the railroad trestle and over the Burlington Ditch back to the entry pit. The mud return line was proposed to reduce the number of vacuum truck trips required to transport drilling fluid from the exit to the entry site for recycling.

A two degree bent sub and a jetting assembly were used to drill the 8-inch diameter pilot bore (Figure 4, Left). The pilot bore was completed in three days, including delays due to inadvertent drilling fluid returns to the ground surface after drilling approximately 400 feet, or two-thirds of the bore. Drilling fluid was seen escaping to the ground surface at the location of the fiber optic line pothole which had been backfilled with grout (Figure 4, Right). The vacuum excavations performed to pothole the fiber optic line were unable to be offset by any appreciable distance from the HDD bore path centerline due to restraints to surface access. There was no increase in the downhole annular fluid pressure prior to the inadvertent drilling fluid returns reaching the ground surface, only a decrease in fluid returns to the entry mud pit. The drilling fluid was immediately contained and removed with a vacuum truck. Pilot bore drilling resumed with the escaping drilling fluid removed using a vacuum truck. A crew member remained stationed at this location for the duration of the HDD in order to monitor and collect drilling fluid migrating to the ground surface.


Figure 4. Pilot Jetting Assembly with Mill Tooth Bit (Left) / Inadvertent Drilling Fluid at Pothole Location (Right).

A single 24-inch diameter forward reaming pass (Figure 5, Left), pushed from the drill rig entry side was performed using a fluted reamer that proved very successful in the alluvial soils. Forward reaming was completed in six days without major incident. Severe cold weather interrupted the drilling operation with a decision to postpone reaming for two days while temperatures were well below freezing. Drill pipe tail string was utilized at all times throughout the forward reaming operation, with an excavator used to provide tension on the drill string at the exit site. Drilling fluid continued to flow from the ground surface at the intermediate inadvertent drilling fluid return pit but was successfully contained and hauled to the mud recycling plant with a vacuum truck.

Figure 5. 24-inch Fluted Reamer (Left) / 10-inch Fusible PVC with Casing Spacers Pushed into the 16-inch Steel Casing Installed via HDD (Right).

The contractor elected to deviate from the initial plan to pull the entire steel casing with FPVC carrier pipe string in one continuous pullback operation. The contractor was allowed to pull the steel casing in two segments with, an intermediate weld performed during pullback. It took just under three hours to perform the intermediate weld, and from beginning of pullback to completion of the casing installation took six hours. After completion of the 16-inch steel casing pullback, the 10-inch FPVC carrier pipe was then pushed into the steel casing with casing spacers used to guide the PVC pipe into place (Figure 5, Right).

CONCLUSIONS

Although the HDD installation was successful, the permit restrictions proved to be onerous during design and construction. The use of plastic casings should be considered by permitting agencies in order to provide more flexibility to the HDD designer, which could reduce project risk and risks to the facility owners. The advancements in plastic pipes are ongoing, and in certain pressure ranges plastic casings can provide the facility owner with assurance that if in operation the carrier pipe were breached, a plastic casing would effectively contain the fluid and prevent damage to the crossing facility. This is certainly the case for the majority of water and wastewater crossings. The same considerations should be made for eliminating the use of casing spacers for HDD crossings that utilize plastic carrier and casing pipes. The use of casing spacers should not be a mandatory requirement for these crossings, but only employed at the pipe system designer's discretion. Eliminating the requirement of using casing spacers would reduce the overall casing diameter, and in turn reduce the required HDD borehole final ream diameter. Reducing the HDD bore diameter decreases overall cost, magnitude of potential ground disturbance, and generally reduces the overall HDD risk profile.

This particular project was successful primarily because of the level of concern and conservatism brought to bear by the design team. In spite of the permitting requirements, the design team consistently favored reduction of construction risk and decreased impacts to interested facility owners over project cost concerns. It should be noted that as a direct result of the analysis and engineering effort expended during the permitting process, FRICO has re-evaluated and subsequently updated its HDD permit restrictions and trenchless installation standards to comport with current advances in trenchless construction. Permitting agencies should be aware that enforcing stringent individual permit requirements at every turn does not inherently result in a low risk or conservative HDD design. As this project demonstrates, quite the opposite can be true when combining many seemingly conservative restrictions that culminate in increased risk. Hopefully this project may serve as an example of why a one-size-fits-all approach to HDD permitting should not continue to be the standard of practice for interested agencies.

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HDD Utilized to Complete Key Crossings for Transmission Lines from New Woodbridge Energy Center

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Abstract

Competitive Power Ventures' Woodbridge Energy Center is a new 700-megawatt, dual energy, high efficiency power generating plant being built in central New Jersey. Located in Woodbridge, New Jersey, the plant will supply electrical power to the New Jersey metropolitan area, meeting the growing demand for power and providing increased reliability for the local grid. When plant construction is complete, power generated at this new facility will be transmitted to Jersey Central Power and Light's existing Raritan Substation, approximately three miles away.

The required three mile transmission alignment between these two locations included some critical crossings, including two environmentally sensitive wetland areas and the longest one, a crossing of the Raritan River. Use of traditional installation methods such as overhead towers and direct burial of conduit were not feasible in these sections of the required alignment. For these three critical locations horizontal directional drilling (HDD) was used to install more than one mile of transmission lines. Each of the three sections included dual parallel installations, resulting in six separate drills of 30-inch casing and conduit pipe installations that would eventually house three 230 kV electrical cables each. In all, over 11,000 feet of HDD installation was completed for these sections.

Fusible polyvinylchloride pipe (FPVCP) was used for both the casing and conduit piping. The 30-inch FPVCP casings housed four 8-inch FPVCP conduits to carry the transmission cables, two 2-inch HDPE conduits for ground and fiber optic lines, and two 3-inch HDPE grout delivery tubes. The entire assembly was grouted in place using an engineered thermal grout for heat dissipation. Local suppliers provided the materials that were tested and used for this project.

This paper will discuss the design and construction elements of the HDD sections, as well as the lessons learned for these key 'underground' sections of the Woodbridge transmission project.

INTRODUCTION

Competitive Power Ventures is (CPV) currently building a new dual energy, high-efficiency generating plant in Central New Jersey. The project, being called the Woodbridge Energy Center (WEC), will produce 700 megawatts of electricity for use in the New Jersey metropolitan area. The WEC will utilize natural gas as the primary energy source and ultra-low sulfur diesel as the secondary energy source to produce electricity. It will generate enough electricity to power more than 600,000 homes helping New Jersey meet its growing demand for energy while also increasing the reliability of New Jersey's energy grid. The project is unique, in that it is situated in a brownfields development area on the site of a former chemical plant, in Woodbridge, New Jersey. In order to tie the new WEC facility into the grid, a transmission alignment was required to connect to Jersey Central Power and Light's Raritan Substation (Raritan Substation), approximately three miles away from the WEC site (see Figure 1).

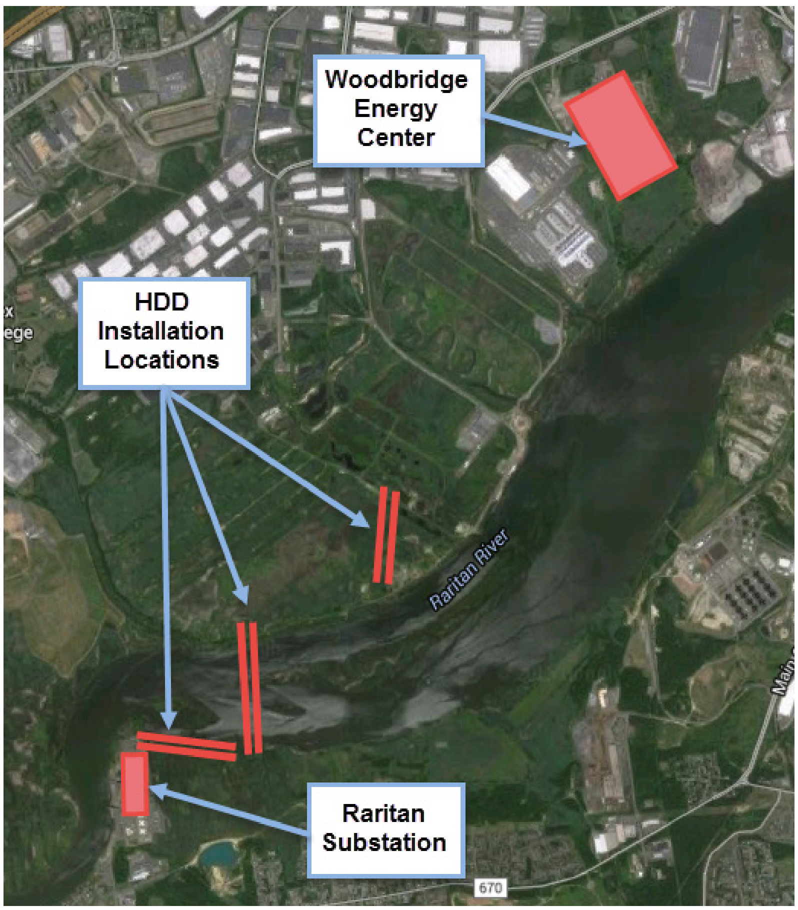

Figure 1. General location map showing location of the HDD portions of the project alignment and the Raritan Substation location.

The three mile separation of the WEC from the Raritan Substation included varied terrain and a river crossing, which complicated the required construction of the transmission cable. Due to the wetlands, river crossing and environmentally sensitive nature of the work, traditional overhead or even direct bury trenched construction was not going to be possible for the entire alignment. For these specific sections, horizontal directional drilling (HDD) technology was used to "underground" the transmission cabling in cased conduits, where project site disturbance was not an option. The entire alignment included sections of overhead, direct bury, and HDD construction in order to meet the project constraints.

Marathon Engineering & Environmental Services prepared the initial alignment drawings that were used to obtain all the local, state and federal permits for this transmission line. While the permit process was underway, several local general contractors prepared design-build proposals for CPV and their construction management firm Kiewit Construction to complete the work. The transmission line portion of the WEC project was ultimately awarded to a joint venture entity, formed specifically for this project, between a Long Island, NY electrical contractor, E-J Electric, and a large New Jersey based civil contractor, Ferreira Construction. This joint venture entity, EJ/Ferreira JV, was well suited for successfully completing the transmission line. E-J/Ferreira JV contracted with the engineering firm, Paulus, Sokolowski & Sartor, LLC, to prepare the construction documents for the traditional overhead and open cut portions of the transmission line. E-J/Ferreira JV then also procured the services of Carson Corporation to design and build the six HDD's associated with the alignment.

PROJECT DESIGN ELEMENTS

The underground portion of the 230 kV transmission lines start about a 1/2 mile behind the New Jersey Convention Center in Edison, NJ and crosses roughly 1-1/2 miles of wetlands as well as the Raritan River. The first HDD location consisted of two parallel bores that crossed 1,500 feet of environmentally sensitive wetlands. The next HDD location included the longest installation lengths for the project, consisting of 2,400 feet crossing under the Raritan River. In between these two HDD locations, E-J/Ferreira JV installed a 1/2 mile of duct bank conduit using direct bury construction methods. The final HDD installation crossed 1,400 feet of environmentally sensitive wetlands and tied the Raritan River crossing into the Raritan Substation (see Figure 1).

The HDD portions of the project included three stretches of dual casing and conduit installations. The total alignment distance completed for the project was approximately one mile, or one third of the project length. Since each HDD segment required the installation of two casings and conduit bundles, the total length of the HDD installations required was over 11,000 feet, or approximately two miles.

Horizontal directional drilling of electrical cable is typically more expensive than traditional overhead lines, and it is imperative to maximize electrical efficiency or ampacity for the underground portion of the transmission lines. The use of plastic casing and conduit, such as polyethylene or polyvinylchloride in place of steel reduces ampacity loss and the use of an engineered thermal grout helps dissipate heat also helping with ampacity loss and extending cable life.

The Carson Corporation elected to use fusible polyvinylchloride pipe (FPVCP) as the casing and primary conduit material for several reasons. First, the tensile capacity of FPVCP provides for a thinner pipe wall for the same buckling and deflection resistance as other thermoplastic pipe options currently available. This means that the overall borehole size could be reduced by using FPVCP, but still provide the required casing and conduit inner diameter for the cable and thermal grout design. Reducing diameter and wall thickness also had tangible value for the HDD process, by reducing the size and weight of both the casing and the conduit, it lowered the risk and cost. The final design for the HDD bores included a 30-inch FPVCP casing, four 8-inch FPVCP conduits, two 2-inch high density polyethylene (HDPE) conduits for the ground cable and a fiber optic line, and two 3-inch HDPE grout tubes. Underground Devices, Inc. custom designed and manufactured casing spacers for the project that held the conduit bundle in the appropriate configuration and also provided wheels for reduced friction of the bundle during installation (see Figure 2).

Figure 2. Final conduit bundle assembly, showing the (4) 8-inch FPVCP conduits, the (4) potential 2-inch HDPE conduits (only two used), the (2) 3-inch HDPE grout tubes, as well as the casing spacers and how the configuration was banded together.

HDD CONSTRUCTION DETAILS

Construction of the HDD segments of the alignment began in early spring, 2014. Required completion of these segments was on a very tight construction schedule, requiring the entire transmission line to be completed and tested by September, 2014. To meet this schedule, the HDD's, overhead cable and direct bury work would all need to take place on the project site simultaneously. Before any of the actual drilling work could start, over two miles worth of timber mats needed to be installed, because much of the HDD work areas were in soft, swamp-like conditions. Timber mats created a stable platform to support the drilling equipment and a location to stage the assembled casing and conduit pipe prior to pull back. A 54-inch steel conductor barrel was constructed at each entry location prior to commencing with the actual drilling activities. A pneumatic hammer, supplied by TT Technologies, Inc., was used to drive 150 feet of casing into the ground at a 12 degree angle. This prevented the soft ground near the surface from collapse during the HDD operations. Once the 54-inch steel conductor barrels were installed for the first pair of HDD alignments, three maxi-sized drill rigs were mobilized to the project site. All three drill rigs are American Augers products, models DD140, DD440 and DD1100. The DD1100 is the largest, with over 1,000,000 pounds of pull back force capability (see Figure 3). To meet the demands of the schedule, drilling took place two bores at a time. Following the drill plan designed by Carson Corporation, a 12-inch pilot hole from entry to exit was created which set the alignment of the bore hole. This was followed by several reaming operations that successively enlarged the hole from 12-inch to 24-inch, 24-inch to 36-inch, and ultimately 36-inch to 48-inch. After the 48-inch hole was cut, Carson Corporation preformed one final "swab" pass to ensure the integrity of the bore hole, sweep any remaining cuttings and excavated material from the borehole, and stage clean drilling fluid for the final pull-in of the casing pipe.

Figure 3. The largest directional drill rig used for the project, an American Augers DD1100.

While the drilling operations were ongoing, Underground Solutions, Inc. was assembling and de-beading the 30-inch FPVCP casing pipe. Sections of 30-inch

FPVCP were thermally butt-fused together into one continuous length of pipe. These lengths were staged beyond and in line with the exit hole of each bore. Each fused joint was internally de-beaded, whereby the raised portion of fusible material that is left after the joints are completed is removed with a mechanical cutter. This was necessary to ensure the wheels on the casing spacers for conduit bundle would not get hung up during the conduit bundle installation. The entire string of 30-inch FPVCP was pressurized with 5 psi of compressed air and pneumatically tested for 30 minutes prior to installation to ensure there was no vandalism or damage to the pipe string prior to installation.

The timing of the construction was orchestrated so that immediately following the swab pass, the 30-inch FPVCP was ready to be installed. Using a mechanical pulling head attached to the 30-inch FPVCP, installation of the entire string took less than 200,000 lbs of force (see Figure 4). To reduce the amount of pull back force needed, the casing pipe was ballasted or filled with water as it entered the bore hole. This acts to reduce the upward buoyant force created by the pipe as it moves through the dense drilling slurry, reducing the frictional forces generated as it is pulled through the bore. All of the HDD bores were completed in this same fashion.

Figure 4. Typical 30-inch FPVCP casing insertion shown with aerial support provided to guide the FPVCP into the required angle of the drill exit.

Each set of bores presented a unique set of challenges. The first set of bores contained the unknown of what this area would entail in terms of actual drilling geology. Although, relatively good geotechnical information was available to characterize the geology, the most valuable geotechnical information is always gathered from the actual drilling. The second set of bores, across the Raritan River, were the longest and had to penetrate a known rock outcropping. Using the information obtained during the first set of bores and some specialized rock tooling, the Raritan River crossings were completed as designed. The final set of bores, which set up to be the easiest at the project start, wound up being the most challenging. For the first two set of bores, there was a 40 foot wide easement to work in. These bores were designed and installed with approximately 30 feet of separation. For the final set of bores, the easement was only 22 feet wide. Further complicating this matter was the fact that the cable manufacturer, Taihan Electric Wire Co., LTD., mandated a 19 foot separation between the bores to prevent the cables from overheating. A sophisticated wire tracking and guidance system that is able to precisely locate the cutting head was used to provide the required separation between the bores while not extending beyond the easement.

After successful installation of the 30-inch FPVCP casing for all six HDD’s, all the large drill rigs demobilized and the bundle installations started. Each 30-inch FPVCP casing would house four, 8-inch FPVCP conduits for the 230kV conductors, including three phases and a spare conduit, two 2-inch HDPE conduits for the ground conductor and a fiber optic communication line and three 3-inch HDPE pipes to serve as the grout delivery system. As with the 30-inch casing, four strings of 8-inch FPVCP were fused, internally de-beaded and staged for installation near the 30-inch casing. A similar process was also performed for the 3-inch HPDE grout delivery pipes. However, since these pipes were sacrificial for the grouting process, there was no need to de-bead each fused joint. Conversely, the 2-inch conduits needed for the ground and communication lines required a smooth internal wall. Using HDPE as the material for these conduits meant that it could be sourced on reels which matched the required length of each bore. Therefore, no joining or de-beading of the 2-inch conduits was required.

It was estimated that approximately 10,000 lbs of pull back force would be required to install the conduit bundle within the 30-inch casing. Instead of using a much larger drill rig, a Vermeer 10x15 directional drill rig with over 10,000 lbs of pull back capacity was used to more closely match the expected pull force estimates. The Vermeer 10x15 drill steel was threaded through the 30-inch casing and each conduit was independently attached to the drill steel. All the conduits were bundled together using stainless steel banding and custom fabricated casing spacers, one every five feet. With just the Vermeer drill rig and a small excavator on the insertion side to help guide the bundle into the 30-inch casing, each bundle was pulled into their respective casing with very little effort. The actual pull back force needed to install the entire bundle for any given insertion never exceeded 1,000 lbs as registered at the drill rig.

THERMAL GROUT DESIGN AND CONSTRUCTION

The purpose of thermally grouting underground installed electrical conduits is to remove heat generated by the transmission of electric power. Heat generated during electrical transmission increases the resistance and thus increases power loss. Additionally, removing heat effectively increases the lifespan of the cable insulation and the cable itself. Overhead wires have constant air cooling. Underground, there is no air circulation, thus another method of heat removal is required. Thermal grouts permit the transfer of heat from the cables into the surrounding soil. The science behind thermal grouting has been around for several decades, but long distance thermal grouting did not become a reality until approximately 2006 (Dickes, 2007; Irani et al., 2007; Dickes and Parmar, 2008).

The advent and increased use of cement admixture technology opened the doors to improved control and behavior of cementitious products. Extended working times, improved flow characteristics and improved grout stability proved very advantageous for the WEC project.

The six, long, cased HDD sections for the WEC project would be comprised of a very tight conduit bundle, meaning that there would not be a lot of space available between the conduits and casing for the grout to flow and fill all voids. This makes the project difficult in applying thermal grout technology. The tight bundle configuration posed two distinct issues. First, there would only be two small, 3-inch grout conduits from each end to work with. Second, the tight bundle arrangements only provided narrow grout flow paths to properly distribute the grout mix. This project would require a very fluid, homogeneous grout to be developed and applied.

Three primary factors need to be considered during thermal grout development. The first is the required, final thermal properties which must meet or exceed the required performance criteria. The second is constructability or the ability to deliver the grout and fully encapsulate the conduits. The third and final factor is cost.

Constellation Group, LLC (CGLLC), the grout specialist for the project, undertook the grout development process. Combining admixture knowledge and the use of specialty silica products, an initial thermal grout was developed. The basic grout components are cement, silica, water and admixtures. In terms of conducting heat, silica, including sand, performs the best, followed by cement, then water. Air is not desirable since it is a thermal insulator. Generally, air content per typical standards (ASTM C231, 2014) is a maximum of 2%, but grouts having 0.5%, or less air, are that much better. Less air means more solids and better thermal conductivity.

Thermal grouts have to perform mechanically during the placement phase. Too much silica produces a grout that is difficult to pump and will segregate during placement, or will “sand-block” the grout piping. A proper balance in the mix design must be established for optimum placement performance. Silica products run from coarse or concrete sand to masonry sand, fine natural sands, less than 250 micron, and silica flours, 50-125 micron, with costs increasing as the grain size becomes smaller. For WEC a fine natural sand was selected as meeting the best overall conditions for the project, including balancing the cost of materials. After the grout mix was formulated, samples were submitted to Geotherm, USA (Geotherm) for thermal resistivity testing. Further refinement of the mix design produced a grout with the required thermal resistivity. The required minimum thermal resistivity was 120º C-cm/W, with a preferable requirement of 90-100º C-cm/W. There was no stated moisture content percentage, so zero moisture content was assumed. Grout in an HDD, cased environment does not reach an absolutely dry condition as there is no place for moisture to migrate if the casing is properly sealed. Typical residual moisture levels are estimated at around 6%. The thermal dryout curve shown in Figure 5 shows the thermal resistivity of the grout mix at different moisture levels.

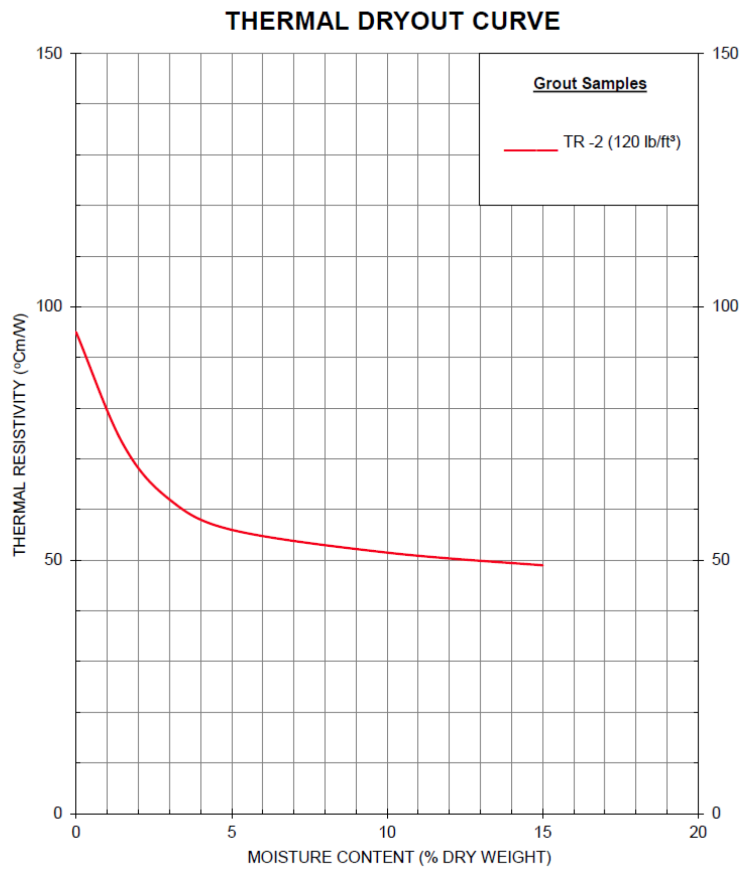

Figure 5. Thermal dryout curve test results for a sample, showing thermal resistivity with varying moisture content. Figure courtesy of Geotherm and CGLLC.

Carson Corporation self-performed the grouting operations with CGLLC's guidance for the actual grout mixing and placement. A locally sourced fine sand was selected for use on this project; however, this product was not compatible with the local ready mix supplier's equipment. Therefore, a hybrid method of batch mixing was utilized as follows:

1.) The ready mix supplier loaded his trucks with the prescribed amount of water, cement and certain admixtures.

2.) When the trucks arrived on site, 15,000 pounds of the specialty fine sand was loaded, along with a final admixture and additional water (see Figure 6).
3.) After the truck was fully loaded, samples were taken to ensure the final mix met or exceeded the designed specific gravity of 128 pounds per cubic foot (pcf). Average unit weights were approximately 130.5 pcf as measured.
4.) In total, over 2,120,000 pounds of fine sand was handled in this manner.

Figure 6. Addition of sand on-site as part of the hybrid grout batching arrangement for the project.

This process created a very fluid grout, with an efflux time (ASTM C939, 2010) of 16 seconds. The use of stabilizers in the mix prevented segregation. The tight configuration of the bundle, further complicated by the small 3-inch nominal grout pipes, limited the grouting process to an average of 24 cubic yards per hour. Pumping pressures, as monitored, were 175 psi on the pressure stroke and near zero on the off stroke. The pressure of 175 psi is misleading, as the pressure gauge was located upstream to the pump hose reduction to the 3-inch grout pipe. Downstream pressure was near zero the entire time, except for the hydrostatic pressure of the grout. Very fluid grouts should not build up around the duct spacers or develop back pressure.

As a standard operating and safety procedure, all conduits were filled with water and pressurized. This serves two purposes. The first is to act as a safety factor against unforeseen grouting pressure spikes; and the second is to act against any heat build-up on the conduits from cement heat of hydration. Many mix designs incorporate heat of hydration control, either through material selection, admixtures, or both.

In total, over 1,350 cubic yards of thermal grout was pumped into the casings. Over 90% of all grout pumped was through the 6 primary, centrally located grout pipes installed in each bundle configuration. Approximately 50 feet near the end of each casing was left ungrouted. This allowed for final excavation and laydown of the casings and conduits to their final elevation. After connection to the respective vaults and tie-ins, the casings were topped off with thermal grout. Grouting was completed in seven, non-consecutive days. The first 1,400 LF casing required two days to complete, due to grout delivery issues with the ready mix supplier. Thereafter, each casing was completed in one day.

PROJECT RESULTS AND CONCLUSIONS

The HDD portion of the transmission alignment was completed on time and within budget. The rest of the transmission alignment portions were also completed within the required timeline. Work on the actual WEC facility continues.

Several records were set on this project in regards to the HDD portions performed and thermal grouting. These are to be considered informal as there is no one agency or institution, other than the personnel performing these tasks, overseeing or maintaining any records. For thermal grout application in sheer volume, this was the largest thermal grouting project to date, including 1,350 cubic yards. This was also the longest total footage of thermal grout installation for one project, including 11,000 feet of thermally grouted casing and conduit bundle.

This project is a good example of how a specialized team of experts and construction professionals can work together to construct a unique project in a successful and efficient manner. The design-build delivery assured that design and construction were in agreement and were realistic.

Long distance thermal grouting is a developing field. Each project has different requirements and only experience, and often times, mock-ups, can provide the correct path. On this project, a trial truckload of grout was batched and dictated a change in delivery quantities. The trial also demonstrated the grout was very fluid and stable and suitable for this project. The hybrid batching plan was also a unique solution to project location, cost and material constraints.

This was the first project in the project team's experience to utilize a fine, natural sand, which proved to be the right choice for this project. However, this may not be the correct material for other projects. Planning well in advance and working with suppliers and contractors is a necessity when it comes to designing thermal grouts to meet the needs of individual projects.

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How to Manufacture an Endless Pipe Onsite

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Abstract

Pipelines have been traditionally constructed in short 20-40 feet (6-12 m) long segments. The pieces are shipped from the factory to the job site and stored on site until they are joined together. This process leads to delays in projects due to the time required to build the pipe segments, and high transportation charges for delivery of the pipes to the job site. Once connected, the joints are a major source of leakage and maintenance expense that continue for the life of the pipeline. The pipe materials require protection against corrosion and the heavy weight of the pipes is a safety concern, making pipeline construction one of the most dangerous trades. In view of the above limitations, the author has developed an onsite-manufactured pipe that allows construction of a virtually endless pipe of any diameter and pressure rating onsite. Unlike conventional pipes, the walls of this pipe are made of a lightweight core that is encapsulated between layers of Carbon or Glass Fiber Reinforced Polymer (FRP). The thickness of the core and the number of layers and type of fibers, i.e. carbon or glass, are determined based on the project loading requirements. This paper focuses on the development of the first prototype of the Mobile Manufacturing Unit (MMU) that was completed in October 2014. Within the MMU, layers of resin-saturated fabrics are wrapped around a mandrel and cured to create the pipe. As the MMU travels along the roadway, it produces a continuous pipe at a rate of 2 miles (3 km) per week. Various aspects of the MMU that were considered and the lessons learned as part of this R&D are presented. A hand-made version of this pipe can be produced with minimal equipment, providing safe drinking water to remote sites and villages worldwide.

INTRODUCTION

Construction of pipes with available technology requires fairly heavy equipment and complex manufacturing facilities. As a result, pipes are constructed in short segments and shipped to the job site, where they are joined together. The result is a pipeline with joints every 20 feet (6 m) or so. These joints are a potential source of leaks, which can inflict significant loss of revenue as well as harm to the environment. For treated water, the problem is so prevalent that the term Non-Revenue Water (NRW) has been globally accepted to refer to the treated water that is lost primarily through leaks. According to a World Bank report, the cost of NRW in 2006 was conservatively estimated at $14 billion (Kingdom, et al. 2006).

For the energy sector, the recent surge of exploration and development of shale gas has increased the demand for pipelines significantly. The Houston Chronicle (Mello 2013) has reported that a shortage of qualified welders has delayed construction of pipelines. The rapid escalation of energy production in shale formations across the U.S. has produced a bonanza of oil, but it has left many states scrambling to handle the natural gas that often flows in large volumes along with the crude. According to a recent article in the Los Angeles Times, the amount of gas flared in the Bakken oil field in North Dakota has nearly tripled since 2011, sending gas worth more than $1 billion a year into the sky (Dave 2014). The primary reason for this waste of energy is the inability to build pipelines quickly.

For large diameter pipes, the transportation costs alone from the plant to the job site add significant expense to the project. Moreover, handling of large pipes is a high-risk task. According to OSHA's records, there were 19 deaths in 2013 in pipeline construction projects; most occurring when the pipes are being loaded onto or unloaded from the trucks or when the pipe is being placed in an open trench (OSHA Fatalities and Catastrophes Report FY2013).

It is our firm belief that the current method of pipe manufacturing is very inefficient and unsustainable; it is only a matter of time before technologies will be developed for on-site manufacturing of pipes. This paper presents one such solution and the lessons that we have learned in pursuit of such a goal.

EARLIER DEVELOPMENTS

In response to the above challenges we started the development of a lightweight honeycomb-FRP pipe that was introduced recently (Ehsani 2012). That pipe (called StifPipe®) uses a similar technology but it is made *by hand* in shorter segments for use in repair of pipes by the slip-lining method. A typical pipe could be constructed according to the following steps:

1. Provide a reusable and easily collapsible mold or mandrel to match the shape and size of the pipe being manufactured; it is best if the mandrel is designed such that its diameter can be adjusted in continuous or small increments;
2. Wrap one or more layers of resin-saturated carbon or glass fabric by hand around the mandrel; the number of layers and type of fiber (carbon or glass) will be determined by our design engineers based on the project pressure requirements;
3. Wrap a layer of a honeycomb sheet on top of the fabric layers;
4. Wrap additional one or two layers of resin-saturated glass fabric by hand around the mandrel;
5. Allow the pipe to cure in ambient temperature (about 12 hours);
6. Collapse the mandrel and remove the finished pipe segment from the mandrel.

This process is fairly simple and we have used it to build pipe segments for repair of gravity and pressure pipes. A 60-ft (18 m) long 24-inch (610mm) corrugated metal culvert was repaired in Mobile, AL (Ehsani 2013) (Fig. 1). To keep the cost down, this pipe was made with glass fabric only. In another application, shown in Fig. 2, seven segments of 4-ft (1.2m) long 48-inch (1220 mm) diameter corroded steel pipes in Avalon Pumping Station, Carson, CA were repaired with this technique (Ehsani and Parsons 2013). The custom pipe segments were manufactured with an outside diameter of 47 inches (1194 mm), to minimize the loss of flow capacity after the repairs. To meet the operating pressure requirements of the plant, this pipe used two layers of carbon FRP on the inside plus two layers of glass FRP as the outer surface.

While the above procedure works perfectly well, its main shortcoming is the speed of construction. These pipe segments are intended to be built in short pieces prior to installation using the slip-lining technique. They require several hours for the pipe to cure on the mandrel. These limitations had to be overcome for an onsite manufactured pipe.

Figure 1. Honeycomb-FRP pipe being made on a mandrel for repair of culvert.

Figure 2. Making and installation of honeycomb-FRP pipe used for repair of pressure pipe.

CONTINUOUS PIPE MANUFACTURING

Considering the relative ease of manufacturing of this pipe, it would be a major achievement if the manufacturing process could be automated to build the pipe in a continuous manner in the field at a fast rate of production. However, to make this transition successfully, there are several design and manufacturing issues that need to be addressed. Each of these challenges are discussed in more detail below.

Figure 3. Collapsible mandrel

Mandrel: The mandrels that we had used had a fixed diameter (Fig. 1) or they required access to the inside of the mandrel to collapse the mandrel and remove the finished pipe (Fig. 2). The automated system must include a mandrel that can be automatically collapsed without access to the inside of the mandrel. One possible design is shown in Fig. 3. The mandrel is made of a tube with a slit along the length. Turnbuckles or electrically-controlled links can be used to reduce the diameter of the mandrel slightly, allowing the finished pipe to be removed. A small overlapping flap along the length of the mandrel can be used to cover the gap that is created by the slit. The mandrel will be supported as a cantilevered arm from one end (Fig. 7b). The finished pipe will come off of the unsupported end of the mandrel (Fig 7a). The operator can control the opening and closing of the mandrel with the switches shown in Fig. 7b.

Surface Finish: A major feature of the mandrel has to be a non-stick surface so that when resin-saturated fabric is cured on the mandrel, it could easily be removed. There are hand-applied or sprayed coatings that can be applied to the surface of the mandrel but these require a fresh application every time a new segment of pipe is being made. This would increase the production time. Other coatings such as Teflon or Mylar sheets can be used also. However, most of these coatings cannot stand the heat that is required for the curing of the FRP. There are similar coatings that could withstand the heat, but these too may not last the full life of the mandrel and periodical re-coating may be necessary. The best solution is a chrome-plated plate. The smooth surface of such a finish is virtually free of any non-uniformities and would allow easy removal of the finished pipe. At the same time, the chrome finish can easily handle the heating of the mandrel during the curing process.

Epoxy: The speed of manufacturing a pipe on site is greatly influenced by the properties of the epoxy being used. The pipes shown in Figs. 1 and 2 were made using a QuakeWrap epoxy which is part of a proprietary system that meets the strict NSF-61 standards for potable water pipes. The resin fully cures in 24 hours in ambient temperature (Epoxy A in Fig. 4). While this feature (i.e. requiring no special curing process) is ideal for repair of pipes and large walls or slabs in a building, the long cure time delays the speed of manufacturing new pipe*s*. Typically, it will take less than 2 minutes to wrap the fabric layers around a 10-ft (3m) long mandrel; after which laborers would have to *wait* while the epoxy cure*s* before removing the finished pipe. Epoxy cure time is the major bottleneck in the production process, so any reduction in this time will significantly impact production speed.

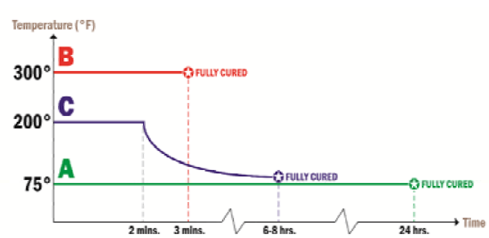

Figure 4. Temperature vs. cure time for epoxies.

After a number of trials and consultations with industry partners, a new resin was selected that fully cures in only 3 minutes if heated to 300F (150C) (Epoxy B in Fig. 2). A further advantage of this resin is that it has a long pot life at ambient temperature. Rolls of fabric can be saturated with resin a day before they are needed. The saturated fabric rolls can be stored next to the MMU in the field and loaded into the MMU to be wrapped around the mandrel. This eliminates the need for mixing resin and saturating fabric in the field, a time-consuming process.

A third category of epoxies shown as Epoxy C in Fig. 2 offer two potential advantages. These epoxies require a shorter time (2 min. vs. 3 min.) and less heat (200F vs. 300F) or (93C vs. 150C) to start the curing process. Once the pipe is removed from the mandrel and the heating source, the pipe will continue to cure at ambient temperature for a few more hours until it is fully cured. Because the pipe is not going to be subjected to any internal or external loads immediately, this type of epoxy appears to be the most advantageous for on-site pipe manufacturing.

Heating Source: As discussed earlier, the epoxy must be heated to initiate the curing process. Several techniques for heating the resin were explored and tested. These included LaminaHeat™ (Fig. 5) which is connected to an electrical circuit and provides a very uniform heated surface. However, because of the time required to raise the temperature in LaminaHeat™ to 300F (150C), it was ruled out. Another promising technology is Variable Frequency Microwave (VFM) by Lambda Technologies (Morrisville, NC) that claims an efficient uniform curing of the resin. Unlike conventional microwave ovens used at homes that operate at a fixed frequency (primarily to excite water molecules), this technique varies the microwave frequencies to ensure that all parts of the subject are heated at the same rate. Samples of fabric and resin were made into a pipe sample cured with VMF in one of Lambda Tech's ovens. The sample looked very good and the epoxy was fully cured. However, a VFM oven based on this technology to fit a pipe would cost around \$100,000. For that reason, this option was ruled out for the time being.

A third system tested was a technology where carbon nanotubes are dispersed in a resin to create an electrically conductive resin. Applying a film of this resin to the inside surface of the mandrel and passing a current through it generates heat that in turn heats the mandrel and the inside layer of the pipe. This technology is viable since the resin bonds to the surface of the mandrel and stays in place (unlike a separate heating film or element that may come apart from the mandrel). The disadvantage is that the resin must be applied manually to the inside surface of the mandrel; making it difficult to apply to smaller diameter mandrels.

Figure. 5. LaminaHeat and propane heater tried in curing the pipe.

To prevent delays in the project, a decision was made to use gas heaters to heat the resin (Fig. 5). For testing purposes, a temporary enclosure was built and the pipe samples were placed inside this enclosure. The pipe was heated using gas heaters both inside and outside the pipe. While this system worked well, it does require supply of propane on the MMU platform. For some remote sites, this could result in additional challenges. For that reason, the use of propane was ruled out.

The heating element used in the first MMU is shown in yellow color in Fig. 7b. This is a clam-shell shaped insulated box that includes electrical heating elements and small fans to circulate the heated air once the shell is closed around the pipe. For the first MMU, the pipe is being heated only from the outside. Considering the diameter of the mandrel for the first MMU 8 in. (200 mm), heating from the outside was sufficient to cure the pipe. As the diameter of the pipe increases, the MMU must be modified to heat the pipe from both inside and outside. The resin/carbon nanotube option discussed above offers a great solution for heating the pipe from inside when larger pipes are being made and the large diameter of the mandrel allows application of this resin.

Interior Finish: Water tightness of the pipe is of course very important. As part of the NSF SBIR Grant, short term hydraulic burst tests were conducted to determine the pressure rating of the pipe (Ehsani 2014). The interior surface of those test pipes were made of two thin sheets of glass veil saturated with resin, the hypothesis being that this combination would create an impervious watertight layer. Tests showed that those specimens started to leak at relatively low pressures (less than 10 psi) due to water seeping through the veil.

Two additional pipe samples were made where a ⅛" (3mm) thick HDPE sheet was wrapped around the mandrel and the edges of this sheet were heat-welded together to create a thin HDPE pipe on the mandrel. Carbon and glass fabric were then wrapped on the outside of this thin HDPE pipe. The result was basically a thin HDPE pipe that derived stiffness and strength from the external FRP layers. A further advantage of such a pipe is that the interior HDPE layer does not bond to the mandrel (unlike a resin-saturated fabric), so removing the finished pipe from the mandrel is much easier. Furthermore, HDPE pipes manufactured in the U.S. by companies such as JM Eagle have been used extensively as water or sewer pipes.

After numerous attempts at creating a good weld at the seams of the HDPE, the pipe samples were made and tested at the Louisiana Tech's Trenchless Technology Center. This sample resisted an internal pressure of 80 psi (5.5 bar) at which time a pinhole in the welded seam of the HDPE started to leak. This leak developed because of poor workmanship. 80 psi (5.5 bar) is more than sufficient for many projects that operate under gravity flow (e.g. culverts and sewer pipes). Improving the quality of the weld will delay or eliminate this mode of failure. However, the welding of the HDPE and automating this process may be too difficult to achieve in the near future. Using one or two layers of chopped glass mat richly saturated with resin will provide a watertight internal surface for the pipe.

Connections and Fittings: The onsite-manufactured pipe described here is best suited as a transmission pipeline where few valves or fittings are needed. However, there will be a need for long segments of the pipe to be connected together. The ends of the pipe can be cut flush and two pipe segments can be externally wrapped with resin saturated carbon or glass fabric to create a longer pipe. Such connections are commonly used in assembly of fiberglass pipes and can produce pressure-rated fittings.

While our focus has been on manufacturing the long barrel of the pipe, elbows and fittings can be built by hand using the same technology as we have used to build shorter pipe segments (Figs. 1 and 2). Moreover, steel or fiberglass flanges and fittings from other manufacturers can be inserted into our pipe and secured with the wet layup system; these flanges can then be bolted together by conventional ways.

For sewer pipe or pipes with low operating pressure, many such products are readily available on the market. As shown in Fig. 6, Inserta Tee® provides a three-piece lateral connection consisting of a PVC hub, rubber sleeve, and stainless steel band that can be easily installed on InfinitPipe®. The connection shown here uses a compression fitting and is suitable for gravity flow pipes. Connections and fittings for water and other pressure pipe applications require further development. If necessary, the connection can be externally wrapped with FRP wet layup to increase its pressure rating.

Figure 6. Installation onto InfinitPipe® of a lateral connection made by Inserta Tee®

MOBILE MANUFACTURING UNIT (MMU)

The first prototype of the Mobile Manufacturing Unit (MMU) was completed in October 2014 (Fig. 7). The unit is only 28 ft (8.5 m) long and weighs less than 7000 pounds (3200 kg), so it can fit in a standard container for shipment to the job site. The lightweight MMU can also be mounted on a flatbed trailer and pulled with a small truck in areas where no developed road infrastructure exists.

One operator controls the entire equipment through the switches that are installed new the right end (Fig. 7 b). Rolls of glass or carbon fabric are saturated with resin in advance. A typical roll is 12 inches (300 mm) wide x 100 ft (30 m) long. The rotating hub shown on the left end of the MMU has arms where these saturated rolls are installed. The angle of orientation of these arms can be easily adjusted, resulting in different pitches for the fabrics being wrapped around the mandrel.

The pitch angle and speed of rotational and translational movement of the hub are set by the controls on the right end. As the hub rotates, layers of fabric are wrapped around the mandrel. The hub then comes to a halt, and the heating oven automatically rises and clamps around the recently wrapped fabric. The oven is heated and within three minutes the pipe is fully cured. The operator then collapses the mandrel, and the finished pipe is pushed out to the left, leaving only a small portion of the pipe on the left tip of the mandrel. The process of wrapping starts again, by continuing at the end of the previously completed pipe. This procedure can continue forever creating an endless pipe.

A

(a) (b)

Figure 7. Views from the (a) left and (b) right ends of the first prototype of the Mobile Manufacturing Unit (MMU)

video of the MMU is available on YouTube and can be watched at this link (http://goo.gl/2KAzuD). In this demonstration video, an 8-inch (200 mm) diameter pipe is being built. The operation is stopped after 12 ft (3.6 m) of pipe has been made. The pipe has a pressure rating of 500 psi (34 bar) and weighs less than 2.5 pounds/ft (3.5 kg/m), allowing for easy handling (Fig. 8). The pipe is rigid enough to allow a truck driving over it without damaging the pipe even when the pipe is not embedded in soil. The MMU can produce pipe at a rate of 2 miles (3 km) per week.

Figure 8. First sample of onsite-manufactured pipe is very light and easy to handle.

The pipe can be either buried directly in a trench or it can be used to slip-line existing pipes. The method of manufacturing of this pipe is so versatile that allows the designer to change the design along the length of the pipeline. The pressure rating of the pipe is determined by the number of inside layers of carbon FRP fabric that are positioned in the hoop direction. As shown in Fig. 9, for example, when a pipe moves along a steep hill, the number of layers can easily be reduced as the pressure in the pipe is reduced due to change in elevation. Similarly, when a portion of the pipe has to span a crossing, additional layers of carbon can be applied in the longitudinal direction to increase the flexural strength of the pipe – acting as a beam.

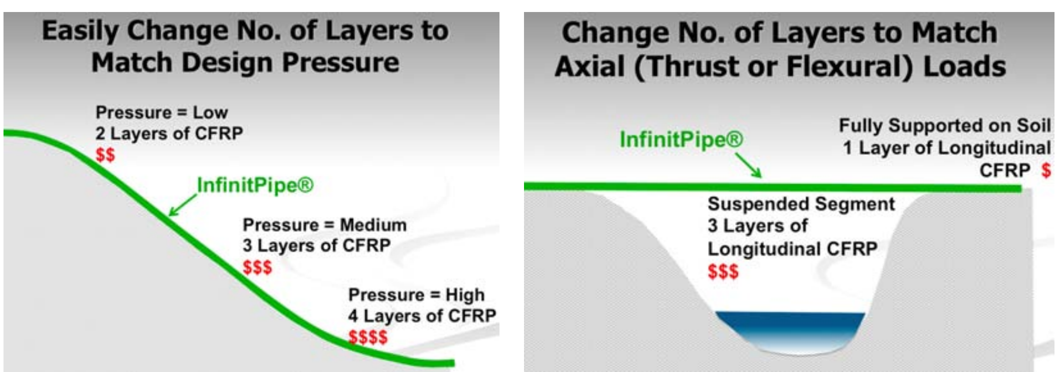

Figure 9. Design of onsite-manufactured pipe can be easily changed to accommodate changes in stresses along the pipeline.

The overall stiffness of the pipe can similarly be modified. For example, for slip-lining a subsea pipeline, it is possible to make a very strong yet semi-flexible pipe that can be pulled into the host pipe as the pipe is made on shore. Such a liner can be designed to accommodate the sweeping angle changes that may be present in the host pipe. Yet the liner/pipe is so light that it will be nearly buoyant in water, requiring little jacking force to pull it into the deteriorated host pipe. We have been contacted by a few clients for such retrofit applications.

CONCLUSIONS

The results of a long term R&D process by the author has led to the development of a new type of pipe that can be manufactured onsite in an endless fashion. The lightweight pipe is non-corroding and can be designed to resist any internal pressure. The unique use of the materials make this pipe very economical. Depending on the diameter of the pipe, one container of raw materials can be shipped to produce over a mile of pipe in a remote site.

While the first prototype of the Mobile manufacturing Unit has been developed and is operational, there are many improvements that can be made on this model. Nevertheless, there is little doubt that such a technology can revolutionize the pipeline manufacturing industry by reducing cost and delivery time, while producing a non-corroding pipe with few joints to leak.

ACKNOWLEDGEMENT

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Hitting the Bulls-Eye:

How to Cut-In a 108" Outlet to a 108" Vertical Shaft 230' Beneath a Lake

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Abstract
Lake Travis is located northwest of the City of Austin, Texas. Due to explosive population growth in the area, the Austin Water Plant No. 4 was designed and constructed in the "hill country" to pull water out of the lake through an intake vertical shaft and tie-in the tunnel located approximately 230' underneath the lake. The project was designed and bid with one contractor doing the shaft and another installing the tunnel and Cut-In Outlet. This paper discusses from a manufacturer and specialty installation perspective how and why the cut-in outlet was required for this installation between the Intake Shaft and Tunnel. Also discussed is how a standard AWWA Manual M11 Design Procedure for Crotch-Plated Fittings in the original bid documents was not applicable for this installation. The paper will go into the design procedure and methodology used and discusses the changes that took place between the design and approval process and the final installation. The first half of this paper will discuss the Pipe Manufacturer's Perspective including the design, bid and approval process for this installation. The second half of this paper will be the Specialty Installation Perspective which will discuss the installation process including the outlet fit-up, welding and NDE practices used for this installation.

OVERVIEW
The Austin Water Treatment Plant No. 4 is located northwest of Austin, Texas. This 300 MGD plant will pull water out of Lake Travis and supply it to the fast-growing area of northwest Austin.

Figure 1 illustrates the hydraulic profile schematic where water is being moved from the lake into the intake screens, then to the raw water intake tunnel. Next is the raw water pump station. Water is then moved from the raw water pump station to the water treatment plant via the raw water transmission main. Total costs for the project is estimated to be around $359 Million, with around $15 Million estimated for the raw water transmission main, including the tunnel installation.

PIPE MANUFACTURER'S PERSPECTIVE

The water treatment plant intake consists of 108" Steel Pipe Intake Screens and Headers that tie into the 108" Vertical Intake Shaft. This shaft drops around 230 vertical feet. There, the

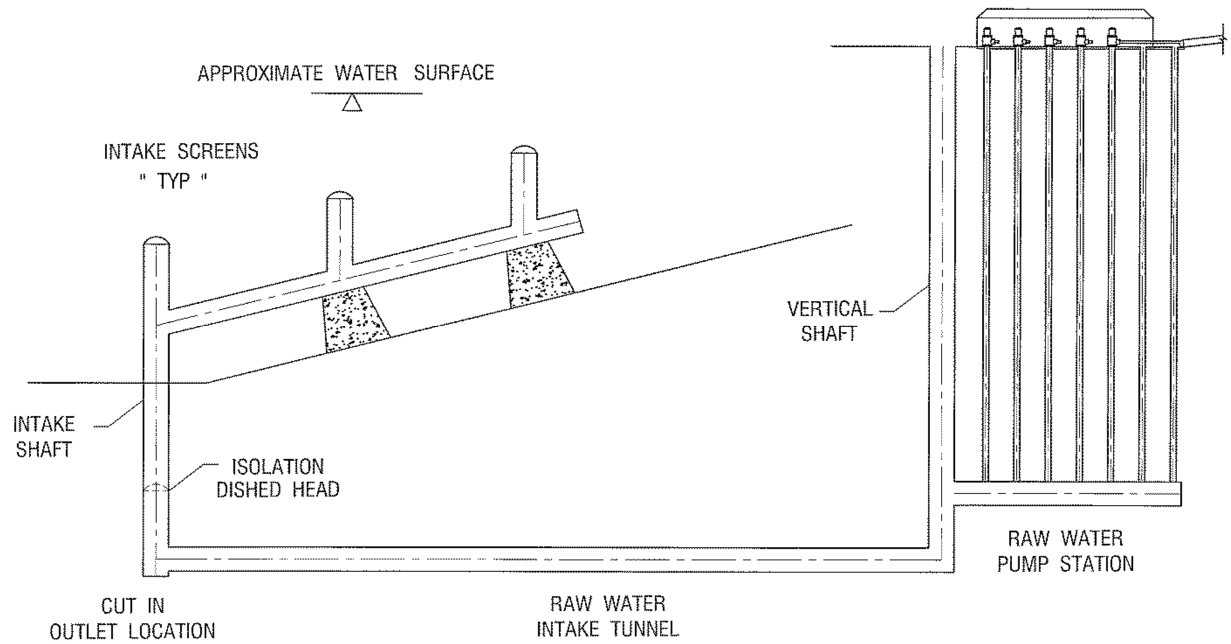

Figure 1. Hydraulic Profile Schematic for Austin Water Treatment Plant No. 4.

connection was made between the Raw Water Vertical Intake Shaft and the Raw Water Intake Tunnel around elevation 450 feet. The field-installed 108" outlet is attached to the 108" vertical intake shaft. Figure 2 shows the location where the outlet will be cut into the vertical shaft. The consulting engineer specified a field cut-in outlet instead of a fabricated tee for a variety of reasons. First, if a fabricated outlet was already installed on the shaft, it would have to be perfectly lined up with the tunnel. Since the shaft was installed first, one could not be 100 percent sure of perfect alignment and if the alignment was off a few degrees, the modifications required would be difficult at best and very expensive. Next, the Raw Water Intake Shaft was installed by a Sub-Contractor to the General Contractor who installed of the Raw Water Intake Tunnel. As stated previously, project sequencing required the Raw Water Intake Shaft to be installed before the Raw Water Intake Tunnel was completed.

Installation Location
As can be seen from Figure 2, the work location was quite compact. There would be no room for any layout or welding on a flat surface.

Figure 2. Field Cut-In Outlet Location around Elevation 450 feet.

AWWA M11 Design
The project specifications called for all fittings to be designed per the AWWA Manual M11- Steel Water Pipe: A Guide for Design and Installation, Manual of Water Supply Practices, Fourth Edition (AWWA 2004). Chapter 13, Supplementary Design Data and Detail outlines the procedure for reinforcement of fittings. Equation 13-1in the above- referenced section, also shown herein as Equation 1, is used to determine the type of reinforcement. The value calculated is called the Pressure-Diameter Value, or PDV. The PDV is based on the ratio of the branch diameter (d) to the main pipe diameter (D) and on the angle of the outlet to the centerline of the main. This value is used in determining the type of reinforcement to be used. Types of reinforcement are collars, wrappers and crotch plates. AWWA Manual M11states that for PDV values less than 6,000, either collar or wrapper reinforcement can be used. For PDV values above 6,000, crotch plate reinforcement is to be used.

(Equation 1) $\text{PDV} = \frac{Pd^2}{DSin^2\Delta}$ (All values are in US customary units)

Where P = Design Pressure (from Hydraulic Profile), in psi
d = Branch OD, in inches
D = Main Pipe OD, in inches
Δ = Outlet Angle, in degrees
Main Pipe and Branch OD =110 ½”
Design Pressure = 150 psi
Delta = 90 Degrees

The PDV for this outlet is 16,575. According to the AWWA M11 Guidelines, this application would require a 3” thick crotch plate type reinforcement with a depth of plate, dw and db of around 70” and a width of plate, dt around 23”, resulting in the fitting being over 10 feet long and nearly 13 feet wide. After analyzing the proposed crotch plates, the dimensions would require that the plates be build in half sections and welded together at the cut-in tee location. Figure 3 shows a butt weld seam on a similar-sized crotch plate from another project where the two halves of the crotch plate were joined together. For this project, the plates would have to have been double-beveled to achieve a complete joint penetration weld, which would require welding on both sides of the plates. This is easily done in the shop but not so in the field, especially at this location. Next, the type of welding would be key. In the shop, this welding can be accomplished using a Submerged Arc Welding Process (SAW). The number of passes to weld just one side of a 3” plate is around 14 to 16. The same has to be accomplished on the backside as well. In the field, the process would likely have been accomplished by flux-core arc welding (FCAW), which would take substantially more time to complete. The ability to rotate the plate over would not be easily or safely done in the tunnel. These welds would require stress-relieving, which would also not be practical given the location. Additionally, the contractor confirmed that the Vertical Shaft would be used as the installation shaft for the cut-in tee and all crotch plates would have to pass through the Intake Tunnel and Shaft, which is around 120” in diameter. The dimensions of each proposed crotch plate were over 124” long and 154” wide, which when installed could have required additional excavation around the shaft, which the contractor wanted to avoid. Finally, the Non-Destructive Examination (NDE) of these welds would be difficult based upon the location. X-Ray methods could not be used due to safety concerns. The NDE methods chosen were visual inspection (V/T) and ultra-sonic testing (U/T). For all of these reasons, crotch plate reinforcement was not preferred.

The first call with the contractor to discuss this application was regarding how to move the crotch plates down the vertical shaft. Following that discussion, a meeting was arranged with the specialty installation contractor that would be performing this installation. After presenting the issues about field installing crotch plates in such a location, the installer suggested to the prime contractor that the pipe manufacturer consider and propose a reinforcement design that did not require crotch plates.

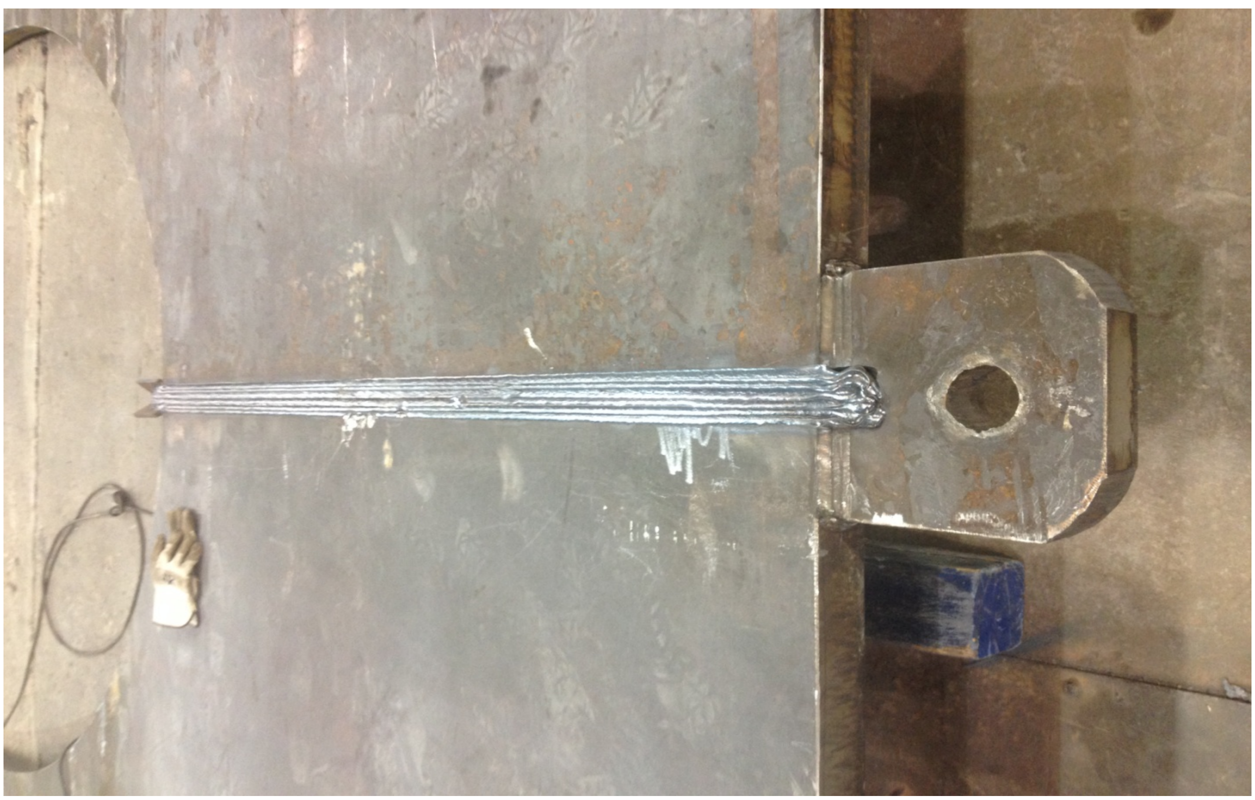

Figure 3. Typical Thick Crotch Plate Seam Weld – Not Chosen for This Project.

Alternative Design Procedure

A couple of design alternates were proposed to the design engineer. One proposal was to change the connection from a cut-in tee to a 90° Elbow. The second proposal, which was accepted by the design engineer, was to design the outlet using ASME Section VIII-Division I – (ASME

2010). This procedure is outlined in the 2010 ASCE Pipeline Conference Paper, "Innovative Design of Large Diameter Fittings for the Lake Fork Interconnect Vault", (Card 2010). This design procedure applies to fittings that do not meet AWWA dimensional requirements or when space limitations prevent to use of crotch plates. This design procedure requires specific material requirements, sets allowable stresses on material and welds, and specifics the safety factors to be used based upon the level of Non-Destructive Examination (NDE) being used. Most importantly for this application, it does not use crotch plates to reinforce the outlet in the pipe. The design uses the thickness of the pipe and outlet and possibly additional collar-type reinforcement to stiffen the opening in the pipe. During consideration of this alternate, the specialty installation sub-contractor asked for the run pipe and outlet to be designed without any additional collar reinforcement.

Based upon the ASME procedures, it was determined that ASTM A516-Grade 70, which has a 70,000 psi minimum tensile stress, would require a base thickness of the outlet to be 1 ¼" for the outlet. This method was proposed and accepted by the design engineer. Below is the manufacturing drawing of the outlet.

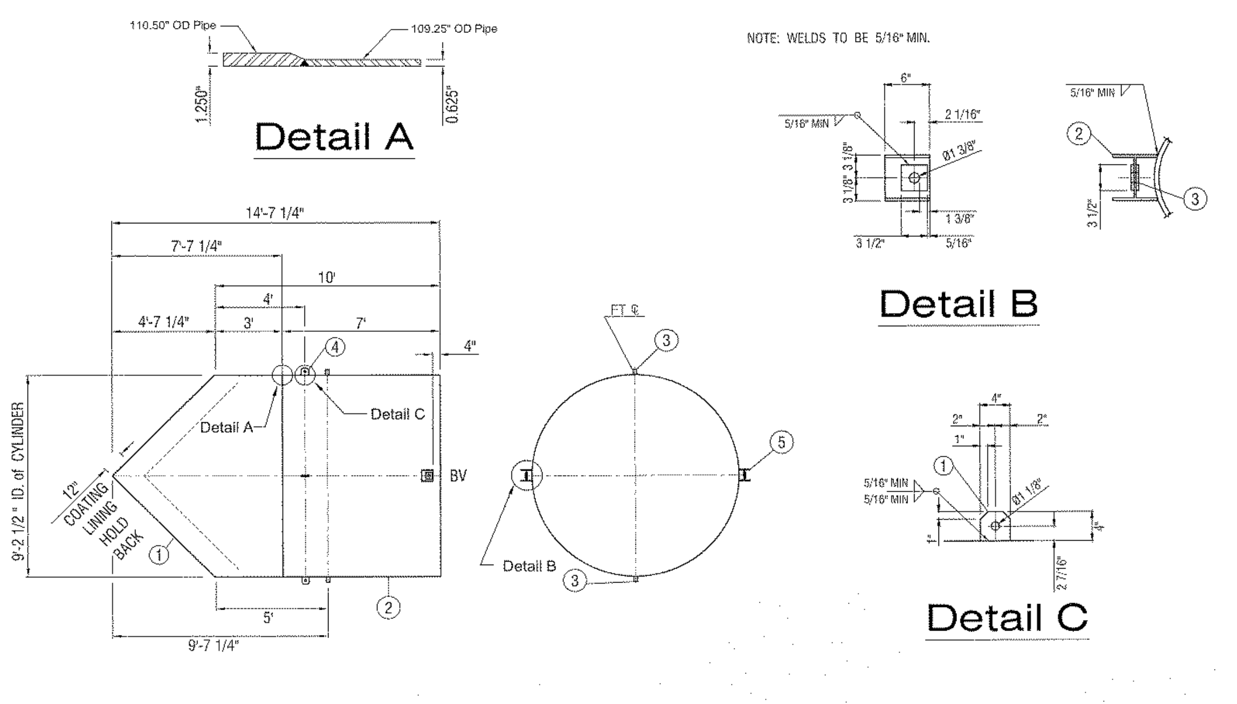

Figure 4. Detail Drawing of Cut-In Outlet.

SPECIALITY INSTALLATION PERSPECTIVE

The Construction Team

Subcontractor National Welding Corporation was responsible to transport the pipe segments inside the tunnel fit and weld the tunnel liner. Obayashi Corporation was the General Contractor of the overall project and self-performed most other key project elements including tunnel excavation, material handling and oversight of all other activities. Northwest Pipe Company prepared the shop drawings, performed all the shop fabrication, and shipped the pipe and fitting materials to site.

Pre-Installation Planning

The team decided that the rail used for the roadheader would remain in place and could be utilized for the pipe installation. The same pipe carrier that was used for the 86” O.D. upper tunnel could support the load of the shorter 110 1/2” O.D. cut-in outlet and transport it to the tie-in location. The carrier would only contact the pipe in four locations (two on the front cart and two on the back cart) due to the difference in diameters, (see Figure 5). However, the increased 1 ¼” wall thickness would support the load of the outlet without deformation.

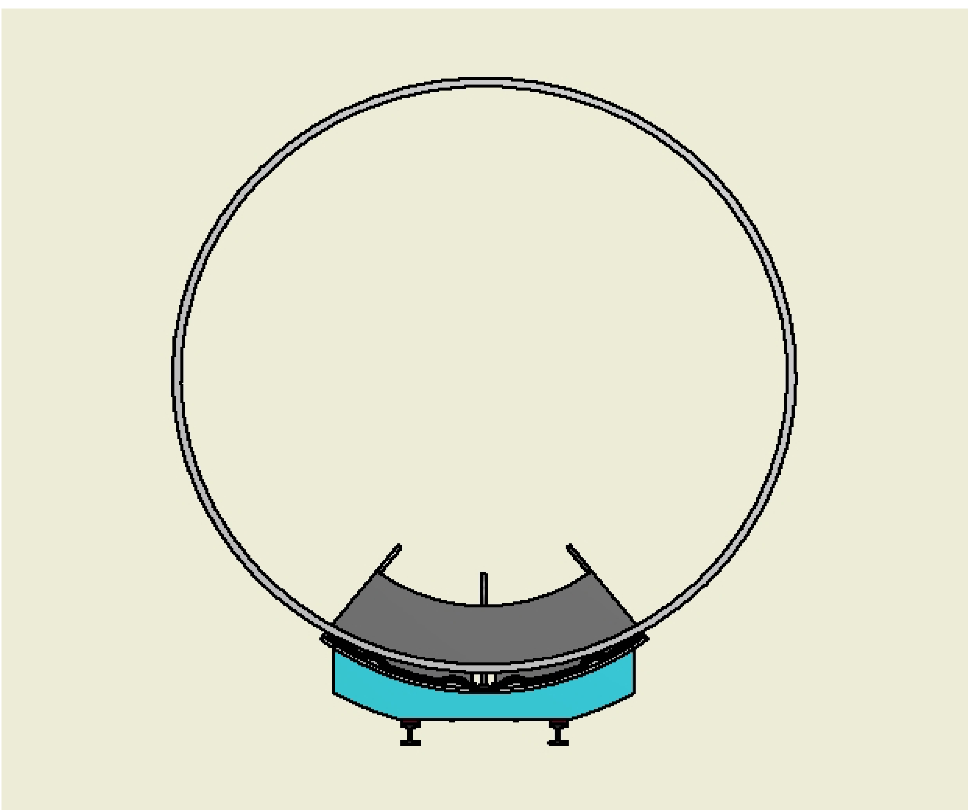

Figure 5. 110 1/2” O.D. Cut-In Outlet on 86” O.D. Pipe Carrier.

The design of the previously used pipe carrier accommodated transportation of the outlet, allowed for clocking or rotation of the outlet segments before engagement at the installation location, allowed lifting the outlet segments to a maximum height of 9” above the carrier, and provided the ability to make minor adjustments side to side to keep the outlet centered inside the tunnel.

Polyurethane coated wheels were used to allow for clocking the outlet segments. Kevlar reinforced rubber coated high-pressure lifting airbags allowed pipe engagement and location adjustments, (see Figures 6a and 6b). The polyurethane coated wheels and rubber coated bags would protect the pipe and outlet polyurethane coating during installation. The lifting bags would use the existing high pressure air connections inside the tunnel for lifting power.

Figures 6a and 6b. Carrier Fabrication and Components.

The pipe carrier and pipe segments were transported to the tunnel installation location using a locomotive with a welded-on 2” trailer ball hitch as an attachment point.

Commencement of Installation

The pipe carrier was lowered down the shaft and connected to the locomotive. Once initial carrier setup was complete, the first outlet section was lowered down the shaft and placed on the pipe carrier, (see Figures 7a and 7b).

Figures 7a and 7b. Carrier Set-up and Outlet Placement.

Since the tie-in location was at the end of a dead end tunnel the bag liner had to be left in place to provide adequate ventilation for welding, cutting, and grinding operations. This made transportation of the pipe and outlet sections very slow since a walker had to walk the full length of the tunnel with a broomstick to keep the bag liner from snagging on the pipe. This was repeated during the movement of all three pipe and outlet sections to assure adequate ventilation for the workmen was not interrupted.

At the cut-in outlet location the rail was cribbed up to the proper elevation. The extra room around the tie-in location was necessary for welder access, (see Figure 8a). The coating was removed from the vertical intake shaft in a 4' x 4' grid pattern to allow initial steel cutting. The pieces were kept to a small size to allow for easy handling, (see Figure 8b).

Figures 8a and 8b. Vertical Intake Shaft and Initial Cylinder Cutting.

The lining was then removed from the interior of the shaft at the tie-in location. This was necessary in order to create a clean oxy-acetylene cut as well as to keep smoke and fumes to a minimum.

With the rough cut-out completed, steel plate dogs were welded to the vertical intake riser. These allowed the outlet to be placed on the side of the intake riser and allowed sliding in and out as needed to trace, cut and refine the field miter preparations without constantly having to adjust the airbags. See Figures 9a and 9b.

Figures 9a and 9b. Rough Cut-out and Trim Cuts.

The pipe was fitted to the intake riser numerous times to scribe, cut and refine the field joint configuration. See Figure 10. A 30-ton push-pull cylinder was attached to the intake riser and the outlet to facilitate this motion, which freed the locomotive for movement between the shaft and intake riser to transport personnel and equipment.

Figure 10. Joint Refinement.

When an acceptable fit up was achieved between the vertical shaft and outlet segment, the air arc gouging method was used to gouge/cut the acceptable joint profile into the weld location. The joint profile had to be adjusted constantly around the circumference of the outlet to provide proper weld joint geometry. See Figures 11a, b and c.

Figures 11a, 11b and 11c. Changing Joint Profile Around Circumference of Outlet.

The liner segments were braced using a combination of 3” x 3” x $^{3}/_{16}$” angle iron and ½” plate “ears” that were field welded to the pipe at spring line. The angle iron members were manufactured at an excessive length to allow for variations in tunnel elevation and were attached to the floor via a steel plate and concrete anchors; see Figures 11a, b and c. The steel plates were laser cut with a pick point attachment that was used to attach a come-along for pipe clocking on the carrier. This bracing method sufficiently supported the pipe for installation purposes. Additional 2” x 2” x ¼” square tubing was installed laterally at the spring line location as well as at the flow line and field top locations at Obayashi Corporation’s request to resist buoyancy forces during annulus grouting. The square tubing needed to be cut to length at location prior to installation to allow for tunnel width and height variations. The liner location, elevation, and minimum tunnel wall clearance tolerance of 6” was verified and documented. See Figures 12a, 12b and 12c.

Figures 12a, 12b and 12c. Pipe Bracing.

Fitting and Welding Operations

Inverter style welding power sources were located at the tie-in location and used the 480 volt power supply that had been used to power the roadheader. The system was capable of powering four welders at one time as well as Air Carbon Arc or Air Arc operations. FCAW (Flux Cored Arc Welding) high production welding procedures were utilized for welding of the circumferential seams to quickly and efficiently complete weld-out of the tunnel within days of completing pipe and outlet installation. The welds received a visual inspection as well as ultrasonic testing by a third party inspector.

CONCLUSION

The supply and installation team is very appreciative of the design engineer accepting the proposed alternative. This project is an excellent example of the "team approach" to problem solving. When specific challenges presented themselves and were brought forward by installers and manufacturers along with design alternatives, the design engineer's receptiveness allowed an alternative approach that improved safety and schedule while avoiding unexpected expense.

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Alternative Pipe Material Choice Provides Trenchless Solution

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Abstract
This is a case study presented on the Big Lake (W14) Gravity Sewer Microtunneling Project in Edmonton, Alberta. The gravity sanitary sewer project was designed as a direct bury application with PVC and RCP pipe. It was expected that the subsurface conditions were to vary considerably within the proposed depth of proposed pipeline. The subsurface conditions within the pipe zone were expected to include water bearing peat, fill, and saturated silt and silty sand, all of which were extremely soft, with blow counts as low as 2. To address these conditions, the design included special bedding and embedment envelops to ensure the installed pipe is adequately supported to prevent pipe settlement and structural failure of the pipe. This design component was more essential to the longevity of the PVC Pipe than the Concrete Pipe. As an alternative, Michels Pipeline proposed to install the pipe by means of microtunneling in lieu of direct bury. In addition they proposed the use of fiberglass jacking pipe for this installation method. The project construction began utilizing 48” diameter FRP jacking pipe, but due to the unfavorable subsurface soil conditions the project was not able to be completed with this pipe material. As a solution to the installation difficulties, the pipe material was changed to 48” FRP Lined Reinforced Concrete Jacking Pipe. This is the first installation of FRP Lined Reinforced Concrete Jacking Pipe in North America.

PROJECT HISTORY

The W14 Sanitary Trunk Sewer is the furthermost upstream stage of the City of Edmonton’s (City) West Edmonton Sanitary Sewer (WESS). WESS consists of large diameter sanitary trunk sewers that will provide both sanitary sewer conveyance and storage for new developments on the western edge of the City between St. Albert Trail and 45th Avenue NW. There are currently 14 stages of WESS, commencing at W14 and terminating near EPCOR’s Gold Bar Wastewater Treatment Plant. WESS is part of the City’ Sanitary Servicing Strategy and is funded through the Sanitary Servicing Strategy Fund (SSSF), which is managed by the City of Edmonton and funded through a partnership between the City and Urban Land Developers.

The W14 Trunk Sewer is comprised of over 7,218 feet of 48” gravity sewer and commences at the intersection of 109th Street NW and 199th Street NW. This section runs south along 199th Street NW crossing Stony Plain Road / 100th Avenue NW and terminates when it discharges to WESS W1 trunk sewer which is located approximately 2000 feet south of 100th Avenue NW.

Once in service, W14 will provide offsite conveyance of sewerage that is generated in the Big Lake Neighborhood and the future Winterburn Industrial Park, which together provides over 3,460 acres of developable land. The Big Lake Neighborhood is located north of Highway 16, east of the City Boundary, south of Big Lake and west of 199th Street NW will be home to over 27,000 Edmontonians. Winterburn Industrial Park is bounded by Highway 16 on the north, 199th Street NW on the east, Stony Plan Road on the south and the City boundary on the west.

GROUND CONDITIONS

Based on the information collected during the project's geotechnical investigation and bore hole program, the generalized stratigraphy along the proposed sewer alignment typically consists of varying thickness of topsoil, clay fill, sand/gravel fill, peat and organic soils. Overlying silty clay was found at depths varying between about 2 feet and 16 feet below ground surface.

It was expected that the subsurface conditions were to vary considerably within the proposed depth of W14. The subsurface conditions within the pipe zone were expected to include water bearing peat, fill, and saturated silt/silty sand. All of which were extremely soft, with blow counts as low as 2 blow per foot.

The project design called for an open trench installation and included two pipe alternatives, PVC Pipe and Concrete Pipe. To address the poor soil conditions, the design included special bedding material and embedment envelop to ensure the installed pipe would be adequately supported to prevent pipe settlement and structural failure of the pipe. This design component was more essential to the longevity of the PVC Pipe than the Concrete Pipe.

HIGH GROUNDWATER TABLE

In addition to the poor soils, the Geotechnical Investigation identified the presence of a high groundwater table over the entire alignment. Due to these conditions, a two stage dewatering plan was developed. The first phase included a trench dewatering/depressurization program prior to excavating the trench. This was specified to prevent base heave of the clay, silt and sand stratum below the pipe bedding. The second phase of the dewatering plan was a trench dewatering program to be implemented where organic materials were found within the trench to ensure the stability of the trench and the safety of the workers.

There was also a pipe buoyancy concern due to the high groundwater table in combination with the fact that the pipe during operation would convey only minimal flows due to the amount of undeveloped lands within the sewer-shed. A requirement for of imported backfill material was included in the design to ensure that the proper ballast is placed above the pipe to counteract the upward buoyant force of the displaced water.

DESIGN

As part of the development of the Big Lake Neighborhood, the Developer was originally tasked with the design and implementation of W14. However, due to the complexity of the project, the risks associated with the installation of W14 via open cut, and higher than expected tender prices, the City elected to further quantify these risks and refine the design to mitigate these risks. The City's Drainage Design and Construction commissioned Stantec to undertake the refinement of the open cut design to address the following risks:

- Potential for high sulfide concentration and the risks of future corrosion;
- Poor soil conditions along the entire alignment;
- The use of flexible pipe and the risk of over deflection due to overburden;
- Buoyancy due to high groundwater table;
- Construction coordination with the contractor consortium for the new Stony Plain Road Interchange.

HIGH SULFIDE CONCENTRATION

The flows from the Big Lake area are conveyed to W14 via a 18,370 foot long forcemain. Due to the length of the forcemain, there may be periods of long sewage retention time within the forcemain resulting in sulfide generation. The generation of sulfides within the forcemain increases the risk of odor and corrosion concerns within W14. The existing Big Lake pump station has incorporated chemical treatment systems to manage the risks to the downstream infrastructure, however, the effectiveness of this chemical treatment systems can only be proven under actual operating conditions. Therefore, the design of the W14 included elements to the potential odor complaints and corrosion of the sewer pipe.

To address the potential corrosion and odor control concerns, the focus of the hydraulic design for W14 was to reduce the amount of turbulence at the manholes to reduce the potential release of H_2S. The design included such elements as the reduction of the height of the drops at the manholes. The goal was to also eliminate the formation of hydraulic jumps within pipe segments prior to entering the downstream manhole.

To further reduce the corrosion potential of the pipe within W14, the design team selected pipe material that was resistant to the corrosive attack of sulfuric acid resulting from the high sulfide concentrations. The team selected two pipe materials, concrete and PVC pipe. PVC pipe is naturally chemically resistant to corrosive attack of sulfuric acid; however, the concrete pipe would require a secondary liner to protect it from the sulfuric acid. To protect the concrete pipe, the design included the provisions of a factory installed HDPE Liner that would be installed during pipe manufacturing process. This HDPE liner would provide the corrosion resistance needed for this sanitary sewer application.

BIDDING PROCESS

Once the design was complete, W14 was let out for bid during the summer of 2011. Prior to this, four contractors were pre-qualified for the project based on their previous open cut experience. The contract went out for bid on July 18, 2011 and closed on August 16, 2011. The bid closing date had been extended by two weeks subsequent to the pre-qualified Contractor's request. Among the four pre-qualified contractors, two valid bids were submitted. In addition to their required bid, an alternative bid was submitted by Michels Canada Co. to construct W14 utilizing a microtunneling trenchless method instead of the traditional open cut method as dictated by the design.

The bids received were considerably higher than expected. It was concluded that the higher than expected bids were a result of the significant risks with constructing a trunk sewer within an area of poor soils and high groundwater.

The lowest bid received was the Michels Canada's alternative Bid to construct by means of microtunneling. However, a caveat to this bid was an extension of the construction window from 12 months to 20 months. A construction period of 20 months would provide the City the opportunity to fund the project over a greater period. Potentially a longer construction period may have impacts on the servicing of the new developments.

The Alternate bid by Michels also proposed the use of three materials not incorporated in the original design. The first was a Hobas Centrifugally Cast FRP Jacking Pipe made of unsaturated polyester resin. The second was Permalok steel casing pipe that is joined by interlocking teeth rather than welding. This would be used as a tunnel pipe underneath the roadways. The third was Hobas Centrifugally Cast FRP carrier pipe which would be inserted inside of the Permalok steel casing pipe.

CONTRACT BID AND AWARD

In late fall of 2011, the City of Edmonton issued Michels Canada a conditional acceptance of award based on the approval of the Alberta Transportation Department for the use of Permalok steel casing pipe within the TUC. The contract required a steel casing to be installed beneath Stony Plain Road to protect and house the 1,200mm trunk sewer. As a part of the alternate bid, Michels Canada proposed the use of Permalok steel pipe which is a steel pipe that stabs together rather than using conventional welding to join pipes. Prior to W14, Permalok pipe had not been approved for use by the Alberta Transportation Department in Alberta.

Michels Canada and the City of Edmonton met with the Alberta Transportation Department in mid-December and formally proposed the use of Permalok Pipe within the TUC. After weeks of deliberation, the Alberta Transportation Department approved the use of Permalok steel pipe within the TUC and thus removing the conditions of approval on the award of the project to Michels by microtunneling.

Once all the product approvals were received, on the City issued a notice to proceed with work on the project on February 14th 2012 to Michels Canada.

PRECONSTRUCTION

After the conditional award had been issued to Michels Canada by the City and during the review period by the Alberta Transportation Department with respect to the Permalok Pipe, Michels proposed to relocate the project's design alignment that ran along the shoulder and TUC of 199th to the middle of 199th Street. This revised alignment allowed Michels Canada to construct the associated tunnel work all year round by removing the soft ground staging issue that the original alignment proposed. Additionally, the revised alignment provided greater access for servicing and maintenance by the City once the sewer is commissioned.

During this phase of the project, Michels Canada proposed to the City of Edmonton to install the 7,352 feet of 48“ ID pipe installed by means of pipe jacking using an Akkerman SL52 - Microtunnel Boring Machine (MTBM) in 10 microtunnel drives. This resulted in the construction of 11 Steel Sheet Pile Shafts and accommodated 7 manhole access points. The depth of the W14 sewer line varied from 33 feet below existing grade on the southern end of the project to 12 feet near the northern end of the project near 109th Ave.

SHAFT CONSTRUCTION

Michels Canada installed interlocking steel sheets with the use of an ABI Hydraulic Pile Driving Rig using a vibratory hammer to drive the steel sheets into place. Dewatering wells were installed post steel sheet construction by local subcontractor Summers Drilling. The dewatering wells were commissioned prior to the start of shaft excavation to draw the groundwater down to a depth below the bottom of the steel sheets.

Both the larger (jacking) and the smaller (receiving) shafts were excavated using a long-stick and mini excavators. As excavation progressed down to the design alignment of the tunnel, Michels installed steel wales and steel corner bracing to hold back the steel sheets.

Upon completion of excavations, concrete working floor slabs were poured with great attention to elevation design details. Next, entrance and exit windows were constructed using steel and wooden form work. The window form work was filled with a low strength concrete that the MTBM would cut through upon launch and retrieval to hold back the soft ground outside of the shafts. A steel faceplate for attaching a 25mm rubber launch seal was cast into the entrance and exit windows. Horizontal cuts were made to the sheet piles just beneath the windows to allow the sheet piles to be raised prior to the launch and retrieval of the MTBM. Once the forms were stripped and the concrete cured, a conventional circular rubber gasket was bolted to the entrance or exit window for the MTBM to tunnel through.

With difficult ground conditions along the W14 alignment, the entrance and exit windows allowed Michels to have a water tight seal around the microtunnel pipe (Centrifugally Cast FRP Pipe) at entrance and exit locations and no ground loss was encountered at any of the tunnel shafts. The entrance and exit seals installed were left in place within the shafts rather than removing and reusing these windows or performing a chemical grouting/ground improvement program to stabilize the ground around entry and exit windows to limit ground loss at these locations. This proved to be a much more cost effective approach to managing the risks at the shaft locations.

The construction schedule allowed for six shafts (3 launch and 3 reception) to be constructed in the first year of construction and five shafts (2 launch and 3 reception) to be constructed in year two of the project. Shaft Construction commenced in late April 2012 due to difficulties in obtaining a revised ULA permit to reflect the new alignment. The first six shafts were completed by August 1st 2012. In year two of the project, shaft building commenced in March and the last of the shafts were completed in July 2013.

MICROTUNNELING

Michels Canada performed the microtunneling using an Akkerman SL52 Microtunnel Boring Machine (MTBM) to install the 48” Centrifugally Cast FRP jacking pipe and the 60” Permalok steel casing pipe. Prior to the start of tunneling, Michels shipped the MTBM to the Akkerman facility in Minnesota for refurbishment and a second rear articulation joint to be installed as a contingency should additional steering be required in the projects difficult ground. The rear articulation joint was never activated. Michels Canada utilized an experienced MTBM operator (Mr. Johnie Paul Halkyard) of over 26 years operating TBMs and MTBMs to operate the SL52 in these difficult ground conditions. The Centrifugally Cast FRP jacking pipe and Permalok pipe was jacked into place behind the SL52 MTBM using an Akkerman 840 ton jacking frame. The slurry separation system was manufactured by Michels Corporation and was outfitted with Derrick slurry separation equipment.

Michels approach to the tunneling was to progress from the southern end of the job (low point of the design) tunneling North up 199th Street to 109th Ave. Michels Canada completed 8 of 10 microtunnel drives with the average daily production rates between 40 feet to 60 feet per 10 hour shift. The highest production rate achieved was 120 feet in a 12 hour shift. The shortest tunnel run was approximately 500 feet with the longest run just less than 1000 feet. The tunnel drive lengths were reduced in length to help mitigate the difficult ground conditions on the project. The Centrifugally Cast FRP jacking pipe and the Permalok steel pipe were both jacked into the tunnel alignment behind the MTBM in 20 foot lengths.

Michels Canada completed the first three tunnel runs south of Stony Plain Road jacking the Centrifugally Cast FRP Pipe with an MTBM skin OD of 49.2”. Michels then skinned up the SL52 MTBM to an OD of 60” to install the next two tunnel runs with the Permalok Steel Casing Pipe beneath Stony Plain Road. Upon completion of the steel casing, Michels Canada used the jacking equipment to then install the Centrifugally Cast FRP 48" carrier pipe with the Permalok steel casing. This completed year one of the project with tunnel activities ceasing on December 23rd.

MORE DIFFICULT INSTALLATION CONDITIONS

As the project has progressed, Michels Canada evaluated the existing ground conditions and conducted additional geotechnical investigation with a number of additional boreholes drilled along the project alignment to evaluate areas with peat in or directly above the pipe zone. Upon review and evaluation, Michels Canada made a material change from the Centrifugally Cast FRP jacking pipe to a Flowcrete jacking pipe.

Flowcrete jacking pipe is a concrete pipe with a Filament Wound Glass-Fiber Reinforced liner which is inert to H_2S attack. This pipe has a greater density to offset buoyancy issues anticipated for two of the five final tunnel drives due to the peat zones. Flowcrete jacking pipe had been used in Europe and the Middle East with success, but this was the first attempted installation in North America.

The original direct bury pipe material options included the option to use an Reinforced concrete Pipe with an HDPE liner. The proposal to use Flowcrete Jacking Pipe (Reinforced Concrete Pipe with FRP Liner) was a pipe material using a similar concept, but with some distinct advantages over other lined concrete pipe materials. Below is a list of the advantages that the Flowcrete FRP lined RCP offered over other lined RCP materials. Also, see Table 1 for comparison of lined RCP pipe materials.

- Flowcrete FRP lined RCP has a liner that has enough structural integrity to resist external hydrostatic pressures without any embedded anchors into the RCP.
- Flowcrete FRP lined RCP has its own joint system. This means that field welding of liner joints is not required. This is required on PVC and HDPE lined RCP pipe products.
- Flowcrete FRP lined RCP is capable of being applied in both pressure water and sewer applications. The joint and liner are rated for pressure up to 450 psi without utilizing the structure or the RCP.

Table 1: Summary of Lined RCP Pipe Materials

Liner RCP Product	Corrosion Resistance	Joint Corrosion Protection Required	External Hydrostatic Pressure - Buckling Resistance	Gravity Sewer Application	Pressure Sewer Application
PVC Liner	Yes	Yes	No – Requires Anchors in RCP	Yes	No
HDPE Liner	Yes	Yes	No – Requires Anchors in RCP	Yes	No
FRP Liner	Yes	No	Yes	Yes	Yes

Michels Canada approached the City of Edmonton to review and approve the merits of the Flowcrete jacking pipe and after consultation and submission review, the City approved the pipe for use on the project. The final two tunnel drives were completed utilizing 1400 linear feet of the Flowcrete FRP lined RCP jacking pipe.

LESSONS LEARNED & CONCLUSION

The project was completed on time and within budget. Some of the lessons learned on this project include the following:

- Value Engineering and "thinking out of the box" provided considerable cost savings to the project.
- Alternate Materials and methods provided added value to the project.
- Use of experienced MTBM operators greatly improve the ability to tunnel in such difficult ground conditions.
- Microtunneling can be successfully installed in difficult ground conditions (2 to 10 blow count) on line and grade.
- Microtunneling is and trenchless technology can be a cost effective solution to the traditional open cut installation.

Teamwork in Trenchless Projects: The Martha Lake Gateway Experience

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Abstract

The I-5 / 164th Martha Lake Gateway Sewer and Water Improvement project, completed in November of 2014, provides a new gravity sewer system and water transmission main for the Alderwood Water and Wastewater District (AWWD) in the vicinity of Interstate 5 (I-5) and 164th Street SW just north of Seattle, Washington. An earlier study performed for the District determined that the most desirable method of providing sewer service to the sewer basin would require a 532-foot trenchless crossing under I-5 as well as a 200-foot crossing of 164th Street Sw. This paper examines the trenchless crossings of I-5 and 164th Street SW with respect to the geotechnical conditions and explores how these conditions influenced the design of the crossings as well as the execution of the contract once construction was underway. The first section focuses on project history and design of the crossings and discusses how the soil conditions directed the trenchless evaluation and selection of the trenchless method for each crossing. The second section focuses on the geotechnical baseline report (GBR) and examines how specific baselines were determined through a collaborative process between the District and design consultants. This process allowed the District to apportion the various risks between themselves and the trenchless contractor. The final section discusses the execution of both trenchless crossings and examines how various elements of the GBR were utilized during construction.

HISTORY OF AWWD

Formed in 1931, AWWD was established in the midst of the Great Depression and rapidly rising unemployment. Originally developed as a series of small five and ten-acre farms, the area was sparsely populated. With the completion of Highway 99, the economic reality of the time improved as people could commute more easily to employment centers in Seattle and Everett. Many of the farms were converted from egg production to other crops or were subdivided and sold, leading to the rapid suburbanization of the area. Sanitary sewer services were first provided after adoption of the Sewer Comprehensive Plan in 1966.

Today, the District is responsible for collecting, transporting and treating residential, commercial, and industrial wastewater within the District's 40-square mile wastewater service area that stretches from the border with King County to the south to the City of Everett to the north. Customers are served directly in both incorporated and unincorporated areas, and also indirectly from upstream wastewater systems in southwest Snohomish County.

PROJECT HISTORY

Prior to the Martha Lake Gateway project, commercial, industrial, and residential properties in the area had been on either septic tanks or private lift stations. Several of these properties had agreements signed in the 1970's and 1990's regarding future participation in a local public gravity sewer which would eliminate the need for private lift stations.

A study commissioned by the District in 2002 determined that a gravity sewer installed under I-5 via a trenchless method was the best long-term solution to serve the sewer basin. In 2007, AWWD contracted with Jacobs Engineering to provide design services for the new gravity sewer. The final design of the sewer was initiated in 2007, although there were several starts and stops along the way with several scope expansions, one of which included the addition of a 30-inch water transmission main which would replace an existing, aging water transmission main. Another addition included the extension of the sewer main to the north side of 164th Street SW, a major arterial that resulted in a second trenchless crossing on the project.

The design team selected by the District included Jacobs Engineering as the prime consultant. The Jacobs design team utilized the expertise of several local sub-consultants including Shannon &Wilson, Inc. who provided geotechnical design services and prepared both the Geotechnical Data Report (GDR) and the Geotechnical Baseline Report (GBR). Staheli Trenchless Consultants also provided their expertise in the evaluation and selection of the trenchless methods as well as assistance in the creation of the GBR.

PROJECT OVERVIEW

The Martha Lake Gateway Project is located in southwestern Snohomish County, Washington, on either side of Interstate 5 in the approximate center of the District's wastewater service area. The upstream end of the new sewer main starts in the vicinity of Exit 183 along I-5 at 164th Street SW, approximately 16 miles north of downtown Seattle. After the crossing of 164th Street, the new sewer main continues south for approximately 3,000 feet through commercial and industrial properties as well as public right-of-way before turning to the west and crossing under I-5. On the west side of I-5, the new sewer main continues westward approximately 600 feet where it connects to an existing sewer main. The project includes the installation of a new gravity sewer main, a water transmission main, and a water distribution main. Figure 1 below provides a site map of the project area and shows the extent of the various sewer and water pipelines.

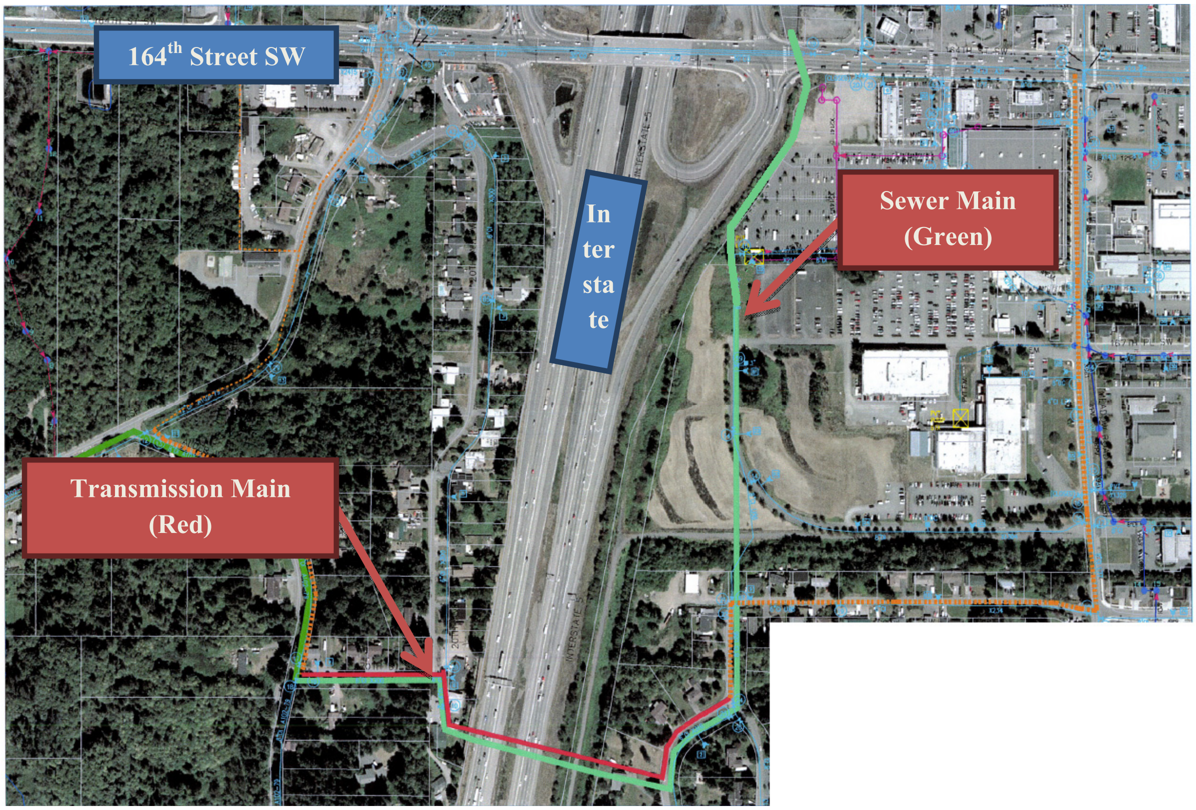


Figure 1. Project Site Map

Final design of the project was completed and it was competitively bid in mid 2013 with mobilization and construction starting in early 2014. The final project that went out to bid included the following elements:

- 532 foot open shield pipe jacked crossing using 64-inch steel casing under I-5 containing a 12-inch sewer main and the 30-inch water transmission main.
- 202 foot auger-bore crossing using 42-inch steel casing pipe under 164th Avenue SW containing an 8-inch sewer main.
- 4,200 total linear feet of 8 to 14 inch sewer main.
- 1,900 total linear feet of 30-inch water transmission main.
- 1,100 total linear feet of 8-inch water main (installed in joint trench with 30-inch transmission main).

As mentioned previously, a 30 inch water transmission main was added to the project scope during the design phase. During construction, the water transmission main was laid dry as it's currently only a piece of multiple projects the District has planned for a regional transmission main. Future transmission main projects will connect this dry line to an existing regional transmission main thus replacing the existing I-5 crossing and allowing it to be abandoned.

GEOLOGICAL CONDITIONS

During the design phase, a total of 12 exploratory borings and 5 test pits were completed at selected locations along the pipeline alignment in order to evaluate geotechnical conditions for both the trenchless and open cut portions of the project. Explorations for the I-5 crossing included a total of four borings and two test pits. Two of the borings and both test pits were located at the shaft locations on each end of the crossing. The remaining two bores were located along the trenchless alignment, one in the median and one just east of the I-5 right-of way. For the 164th Street crossing, two borings were performed, one at each end of the 202 foot crossing. Pump-down recovery tests were also performed at four observations wells in order to determine the hydraulic conductivity of the surrounding soils.

Based on the exploratory borings and test pits, the soil and groundwater conditions at both crossing locations were determined to be similar. Soils along the crossing alignments generally consisted of shallow fill over glacial till. The fill ranged from 3 to 10 feet thick and consisted of very loose to medium dense, silty sand with varying amounts of gravel, clay, and organics. The underlying glacial till consisted of very dense, gray, silty sand to sandy silt with varying amounts of gravel. For both crossings, the upper 5 to 10 feet of the glacial till was found to be weathered and was medium dense to very dense and brown in color. Although not encountered in the existing exploratory borings and test pits, the glacial till is known to contain cobbles and boulders. Based on the proposed depths of the pipelines, it was determined that both trenchless crossings would be constructed entirely in very dense intact glacial till.

Based on observation well measurements, groundwater elevations along both crossings were well above the crown of the tunneled casings. Due to the very dense nature and low permeability of the intact glacial till, it was determined that the groundwater was perched above the intact glacial till in the less dense fill and weathered till. Although this groundwater was determined to be perched, several of the test pits and borings indicated the potential for lenses and layers of sand within the intact glacial till that could contain limited, isolated amounts of groundwater.

In addition to the exploratory borings and test pits, site reconnaissance and observations of excavation activities at an adjacent site near the west end of the I-5 crossing provided valuable information on the presence of cobbles and boulders in glacial till soils similar to those anticipated along both crossings. Eight boulders were encountered in a sanitary sewer trench which was reportedly 1,200 feet long by 4 feet wide by about 10 feet deep. The boulders ranged from 2 to 7 feet in their maximum dimension. Based on this data, it was estimated that one boulder per 222 cubic yards of excavation could be encountered in the glacial till soils. Figure 2 below shows just one of these boulders being measured by the site's excavation contractor.

Figure 2. Excavated boulder at adjacent site

TRENCHLESS EVALUATION PROCESS

Once the geotechnical explorations and observations had been completed, the results were summarized in a geotechnical data report (GDR). Using information in the GDR, several trenchless methods were evaluated for both crossings. Using the evaluation matrices shown below, the various trenchless methods were assessed for compatibility with the soil and groundwater conditions and evaluated against the requirements for the specific crossing. The end result of the trenchless evaluation was the selection of the following preferred methods for each crossing.

Open Shield Pipe Jack Crossing of Interstate 5

Given the dense glacial nature of the soils, length of the crossing, limited access within the I-5 right-of-way, and high probability of encountering cobbles and boulders along the crossing; open shield pipe jacking (OSPJ) was selected for the 532 foot crossing of I-5. As presented in the evaluation matrix (Figure 3 below), the selection of OSPJ was primarily driven by the desire for face access to facilitate the removal of obstructions as the limited access within WSDOT right-of-way eliminated the possibility of a rescue shaft for the majority of the crossing.

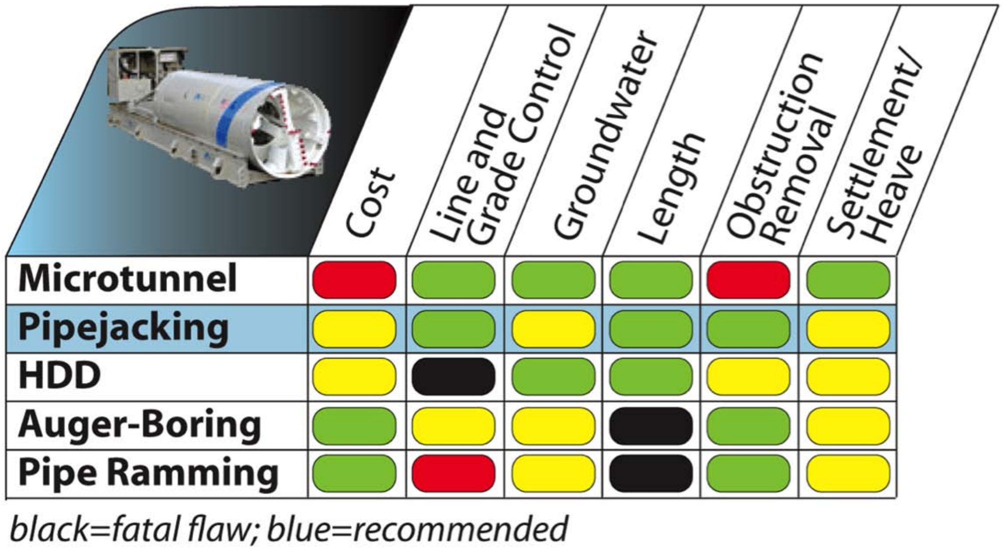


Figure 3. Interstate 5 - Trenchless Evaluation Matrix

Auger-Bore Crossing of 164th Street

Even though the soil conditions for the 164th Street SW crossing were very similar to those of the I-5 crossing, the shorter 202 foot distance was reasonable for an auger-bore crossing. It was therefore not eliminated from consideration for the 164th Street SW crossing during the trenchless evaluation, as it had been for the longer I-5 crossing. Utilizing a larger casing than what was required for the 8-inch sewer crossing allowed man-entry into the bored casing if required to remove obstructions. As shown in Figure 4 below, the ability to access obstructions and the shorter length of the crossing were two of the driving factors in the selection of auger-boring for the 164th Street crossing.

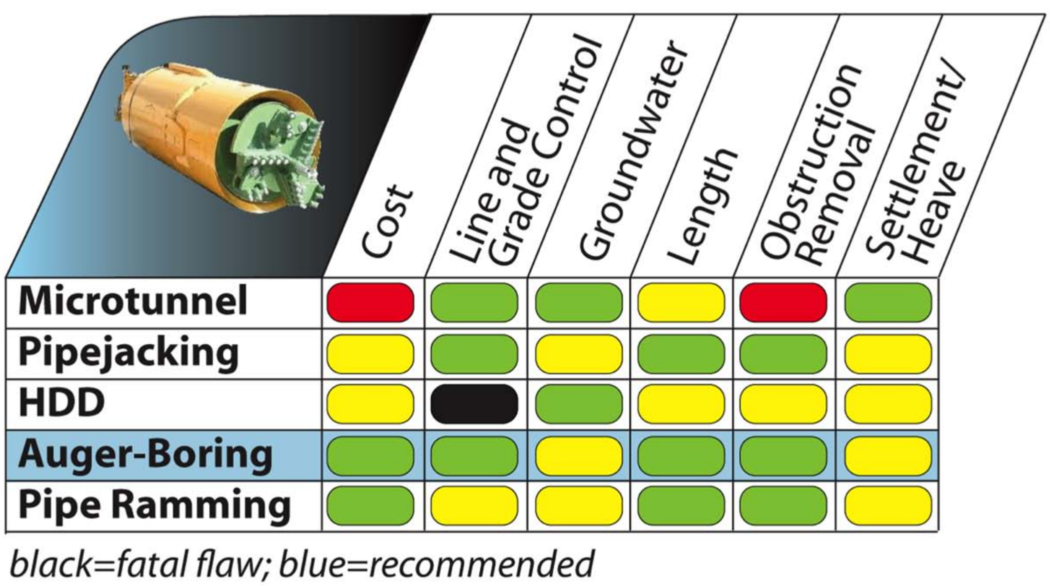


Figure 4. 164th Street - Trenchless Evaluation Matrix

Once the crossing methods had been selected, the project drawings and specifications were modified to reflect the requirements of each particular method. This process included laying out the staging area required at each shaft location, which allowed the limits of temporary construction easements to be determined. Similar baselines were set in the GBR for both crossings, although the number and sizes of obstructions were modified based on the diameter and length of the casing pipe to be installed.

CREATION OF THE GBR

A comprehensive GBR was developed through a collaborative process which included the owner (AWWD), the design engineer (Jacobs), the project geotechnical engineer (Shannon & Wilson), the trenchless consultant (Staheli Trenchless), and the owners' legal counsel. The GBR considered the information gathered during the geotechnical explorations, known regional soil conditions, observations of boulders from the adjacent development, and consideration of potential costs associated with various obstructions. After several rounds of comments and revisions by the entire project team, the following specific baselines were set which defined the quantity of boulders, wood, and groundwater to be anticipated over the length of each crossing:

<u>Interstate 5 - Open Shied Pipe Jack Baselines</u>
(532 LF at 62-inch to 84-inch diameter casing)

- Anticipate that layers and lenses of cohesionless soils will comprise 10 percent by total volume of the glacial till excavated.
- Anticipate cobbles and boulders up to 14 inches will be encountered all along the alignment.
- Anticipate up to 5 boulders ranging in size from 14 to 25 inches.
- Additional payment will be considered when the number of boulders ranging in size from 14 to 25 inches exceeds five and when boulders measure over 25 inches.
- Wood with a maximum dimension greater than 25 inches will be considered for additional payment.
- Anticipate steady-state groundwater inflow into the casing from the tunnel face of 10 gallons per minute (gpm).
- Anticipate four separate areas of higher transient groundwater inflows of 20 gpm lasting one hour.

<u>164th Street - Auger-Bore Baselines</u>
(202 LF of 42-inch diameter casing)

- Anticipate that layers and lenses of cohesionless soils will comprise 10 percent by total volume of the glacial till excavated.
- Anticipate cobbles and boulders up to 1/3 the outer diameter of the casing will be encountered all along the alignment.
- Anticipate up to 2 boulders ranging in size from 1/3 the outer diameter of the casing up to 42 inches.
- Additional payment will be considered when the number of boulders ranging in size from 1/3 the outer diameter of the casing up to 42 inches exceeds two and when boulders measure over 42 inches.

- Wood with a maximum dimension greater than 42 inches will be considered for additional payment.
- Anticipate steady-state groundwater inflow into the casing from the tunnel face of 10 gallons per minute (gpm).
- Anticipate two separate areas of higher transient groundwater inflows of 20 gpm lasting one hour.

It should be noted that in order to be considered for additional payment, all potential obstructions must have first stopped forward progress of the tunneled excavation. For instance, if a 30 inch boulder was encountered during the open shield pipe jack under I-5, it would only be considered for additional payment if it stopped the advance of the pipe jack.

SUMMARY OF CONSTRUCTION

At bid opening in July of 2013, TITAN Earthworks was the selected general contractor. For both trenchless crossings, TITAN used the services of Northwest Boring based in Woodinville, Washington. For both crossings, Northwest boring utilized equipment they owned which had been used successfully on crossings in similar soil and groundwater conditions. Table 1 below summarizes the trenchless equipment Northwest Boring used for each crossing:

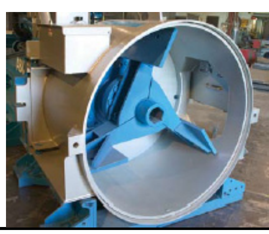
Open Shield Pipe Jack		
	TBM Model	Akkerman WM 66SC
	Machine OD	66"
	Overcut	3/4"
	Jacking System	Akkerman 5000 Series
	Jacking Capacity	400 Tons
Auger-Bore		
	Machine Model	Robbins ABM 48-950
	Net Power	174 hp
	Max. Thrust	954,000 lbs
	Cutting Shoe Dia.	42"

Table 1. Trenchless equipment specifications

As anticipated in the GBR, both trenchless crossings encountered occasional cobbles and boulders. Figures 5 and 6 below show examples of cobbles and boulders encountered during both crossings as seen from inside the tunneled excavation as well as being measured once they had been removed.

Figure 5. Boulder encountered during pipe jacking

Figure 6. Cobble encountered during auger boring

In total, there were four boulders encountered during the I-5 crossing which were determined to be obstructions eligible for additional payment because they exceeded the minimum dimension of 25 inches and stopped forward progress, per the requirements set forth in the GBR. Through the change order process, the contractor negotiated payment for removal of these obstructions. Figure 7 below shows one of these obstructions being removed from the tunneling shaft and being measured on the surface.

Figure 7. Obstruction (rock #3) being removed and measured

CONCLUSION

By the end of substantial construction activities in November of 2014, both crossings were completed successfully. The pipe jacked crossing of Interstate 5 took approximately 45 days to complete from the mobilization of tunneling equipment to the break-out at the reception shaft. The shorter auger-bored crossing of 164th Street was completed in approximately 22 days.

The collaborative process that was utilized to set specific baselines for both crossings provided a starting point for contractors to competitively bid the project. Once the project went to construction, all boulders encountered during the trenchless crossings could be measured and compared against the baseline. The end result was that the owner only paid for those obstructions which significantly impacted the progress of tunneling and the contractor was able to recover those costs not included in his bid price associated with removal of significant obstructions.

Experimental Examination of the Mathematical Model for Predicting the Borehole Pressure during Horizontal Directional Drilling

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Abstract

Predicting the borehole pressure during Horizontal Directional Drilling (HDD) is a significant part of HDD. Borehole stability means that pressure on the bore-face must be less than formation fracture pressure and more than the collapse pressure to avoid fluid losses or borehole breakouts. The proposed research is aimed at an analysis comparison between the mud pressure data collected in the real field and the ones the mathematical model predicted. Then the optimal model will be applied to predict the allowable maximum borehole pressure during HDD. Borehole stability during drilling consists of evaluating the drilling fluid weight to maintain the borehole wall integrity. The tensile failure (hydraulic fracturing) and dog-ear shape breakout are two main failure modes around boreholes during HDD. The cavity expansion model was used to calculate maximum and minimum allowable drilling fluid pressure in a bore. Both 2D and 3D finite element (FE) models of maximum borehole pressure were developed by the Drucker-Prager and Mohr-Coulomb theories using ANSYS Parametric Design Language (APDL) to support the customized parametric study. The result showed the maximum mud pressure closely matched the estimation obtained using the Delft equation in this field experiment for shallow layers within clay. The FE modeling procedure used was able to capture the volumetric compressive behavior of the soil around the borehole.

Introduction

The full-scale field study on a newly developed compaction reamer was conducted at the test site of the Trenchless Technology Center located in the Louisiana Tech University's South Campus. The site had trapezoidal shape with 425 ft. and 435 ft. in longer sides and 150 ft. and 50 ft. in the shorter sides. The boring was initiated from the west side (narrower side) of the field. The natural slope was measured using a level and found to be 3.2% from west to east. The dominating soil type at the site was normally consolidated non-saturated deformable silty-clay with inter-bedded lenses of sand. The groundwater table was well below the invert of the installed pipes. This paper provides an analytical comparison between the real mud pressure data collected by a custom made load cell in the field and the calculated mud pressure from mathematical prediction model.

Field Data Collection

Prior to the commencement of the construction, all buried utilities were located and marked. The proposed drill plan was designed considering the natural ground slope and all possible potential conflicts that may arise. The installation cover was determined to be 8 ft. in order to minimize the risk of frac-out during the installation. The pilot hole was drilled with a standard 4.5 in. drill bit. Then a paddle reamer of 12 in. diameter was used to enlarge the borehole. The back-reaming and product pull operation were performed in a single stage with an assembly shown in Figure 1. The product pipe was fusible PVC pipe with a 9 in. of outer diameter. The load cell attached to the reamer using a swivel connection had a pressure gauge housed in its front face which collected the mud pressure reading during pull back operation. Each installation was designed to be 300 ft. long.

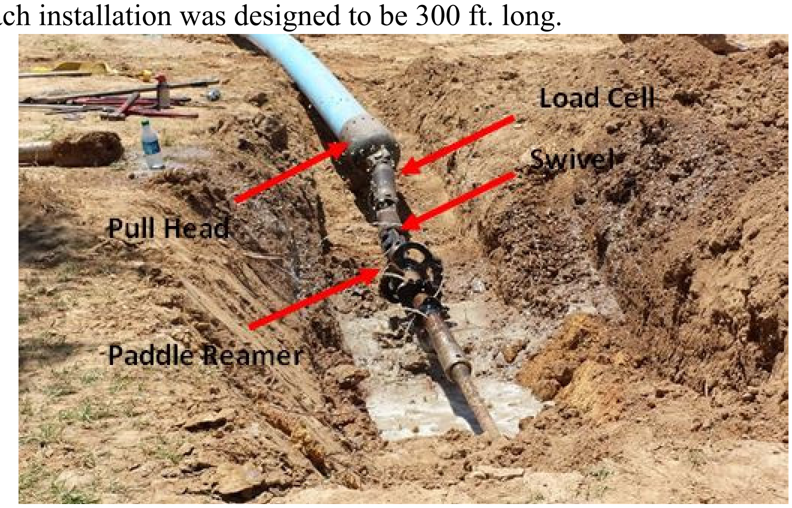

Figure 1. Reamer, Load Cell, and Pull Head Assembly.

The mud pressure data collected during the pull-back operation is plotted against the horizontal distance and shown in Figure 2.

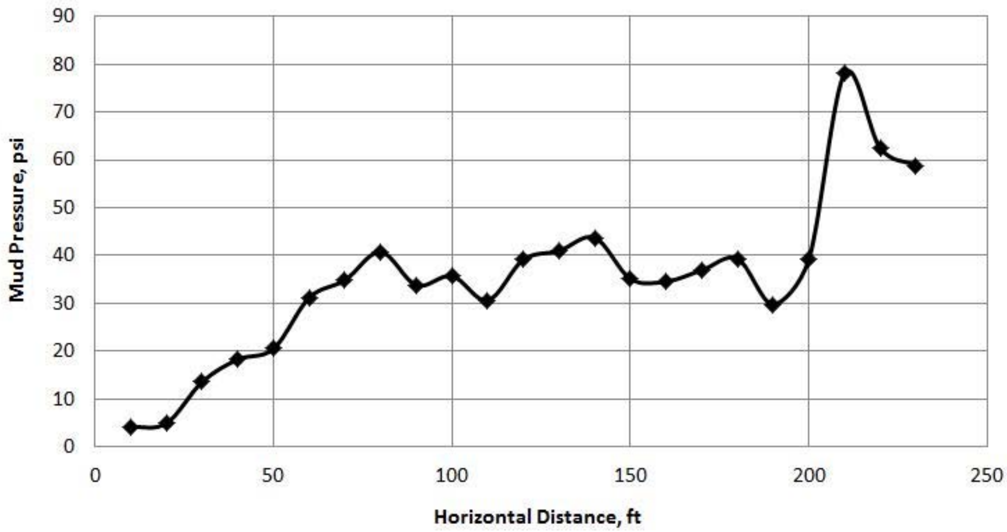

Figure 2. Mud pressure during pull back operation.

Figure 2 shows that the drop in mud pressure readings around horizontal distance marks 80 ft., 150 ft., and 200 ft. was initiated by the formation of a frac-out. Later on, a 2D and 3D ANSYS model will be simulated to check the frac-out locations where the borehole pressure was expected to overcome the overburden pressure.

3. Maximum Drilling Fluid Pressure Theory

Borehole stability during drilling consists of evaluating the drilling fluid weight to maintain the borehole wall integrity. It means that pressure on the bore-face must be less than formation fracture pressure and more than the collapse pressure to avoid fluid losses or borehole breakouts (Amsterdam et al. 2008). These phenomena are associated with tensile failure (Figure 3a) or shear failure (Figure 3b), respectively. In the present case, silt-mode breakout (Figure 3c) was obtained during the HDD drilling. The most common elastic-plastic constitutive models used in borehole stability analysis are shear-failure models such as Drucker-Prager and Mohr-Coulomb, associated with some tensile failure criteria. These models are representative of lower porosity soil behavior. The compaction mechanisms can be predicted by cap models (Fjaer et al, 2008).

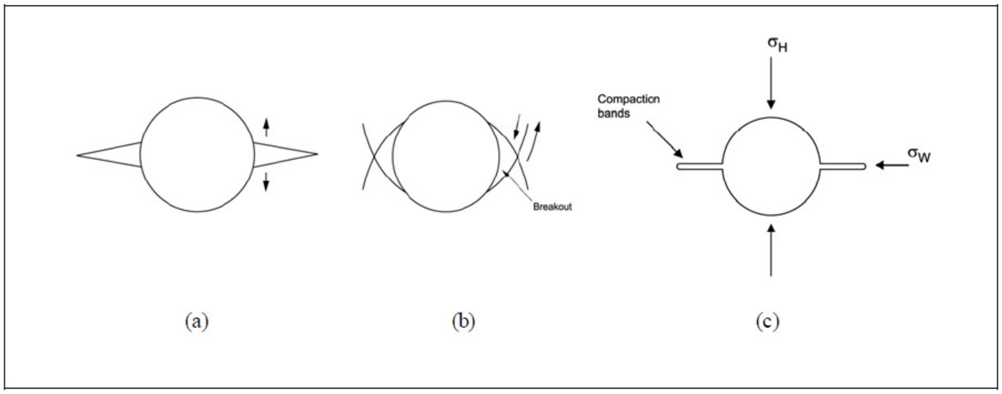

Figure 3. Failure modes around boreholes (a) tensile failure (hydraulic fracturing); (b) dog-ear shape breakout; (c) fracture-like breakout.

An underground soil mass is an equilibrium condition under compressive in-situ stress state which can be decomposed in relation to a Cartesian Coordinates system; a vertical stress parallel to the depth direction, and two horizontal stresses: a major horizontal stress (SH) and a minor horizontal stress (Sh). Changes in these stresses are introduced by the drilling and production operations.

As the bore is drilled and material is removed, a stress relief occurs. If the cavity is not filled with fluid, the equilibrium is attained by tangential stress concentration. The drilling fluid introduces a radial pressure against the borehole wall. This pressure acts as a support to the bore-wall and relieves the generated tangential stress. The stress state around the borehole-wall may vary according to the borehole radius and inclination angle.

This variation depends on many factors, such as the borehole direction related to in-situ stresses, the magnitude of in-situ stress, the rheology of the rocks and borehole geometry. As one moves from the borehole into the formation, the stresses tend to reach the in-situ stresses. According to Rocha and Azevedo (2009), the usual stress configuration around the borehole is: a tangential stress which is the major principal stress and an axial stress which is the intermediate stress. Although in lower depth the axial stress may become the major stress, the radial stress is the minor principal stress.

Maximum Allowable Mud Pressure from Cavity Expansion Method

The cavity expansion model (Allouche et al. 2000), as discussed previously, can be used to calculate maximum allowable drilling fluid pressure in a bore. This is the first step in the evaluation of hydro-fracture risk. The maximum allowable pressure can be expressed as (equ.1&equ.2):

$$Q = \frac{\sigma_o \sin(\varphi) + c\cos(\varphi)}{G} \; [1]$$

$$P_{\max} = [\sigma_o(1+\sin(\varphi)) + c\cos(\varphi) + c\cot(\varphi)][(\frac{R_o}{R_{p.\max}})^2 + Q]^{\frac{-\sin(\varphi)}{1+\sin(\varphi)}} - c\cot(\varphi) \; [2]$$

Where,
$P_{\max}$ = the maximum allowable mud pressure,
u = the initial in-situ pore pressure,
σ_0 = the initial effective stress,
φ = the internal friction angle,
c = the cohesion of the surrounding material,
R_0 = the initial radius of the borehole,
$R_{p,max}$ = the radius of the plastic zone,
G = the shear modulus of the surrounding soils.

All the parameters needed to calculate the maximum drilling fluid pressure and all these inputs are estimated using typical values for anticipated soils, whose references are all listed in table 1. The maximum borehole pressure predicted from the Cavity Expansion theory is plotted in Figure 4 where the maximum mud pressure is to be 41.6 psi.

Table 1. Soil parameters for maximum drilling fluids formula

Parameters	values	Reference
Soil Unit weight above groundwater γ(*lb/ft*^{3})	133	http://www.geotechnicalinfo.com/soil_unit_weight.html
cohesion c (*psi*)	2.9	http://www.geotechdata.info/parameter/cohesion.html
Angle of internal friction φ	30.5°	http://www.stanford.edu/~tyzhu/Documents/Some%20Useful%20Numbers.pdf
Shear modulus G(*psi*)	6670	http://ascelibrary.org/doi/pdf/10.1061/(ASCE)GT.1943-5606.0000887
Clay unit weight(lb/ft^3)	102	http://www.geotechnicalinfo.com/soil_unit_weight.html
Young's modulus E(*psi*)	174	http://www.stanford.edu/~tyzhu/Documents/Some%20Useful%20Numbers.pdf
Poisson's ratio	0.33	http://www.geotechnicalinfo.com/soil_unit_weight.ht

ν		ml
Coefficient of lateral earth pressure at rest K_0	1.0	http://www.geotechnicalinfo.com/soil_unit_weight.html

The frac-out during drilling operation occurred at 90 ft., 110 ft. and 150 ft. from the exit point, which are marked in Figure 4. According to the maximum pressure model from the Cavity Expansion Theory, in non-saturated deformed clay the drilling fluid losses would happen at 80 ft., 100 ft. and 140 ft. in horizontal distance from the exit point, which closely predicted the actual locations of frac-out during directional drilling.

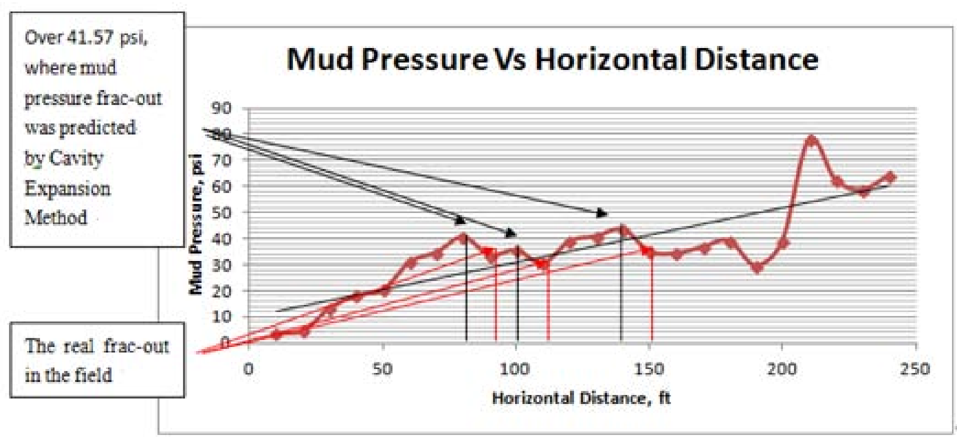

Figure 4. The mud pressure and location of frac-outs which Cavity Expansion Method predicted and field data

The allowable maximum mud pressure due to the hydraulic fracturing can also be estimated using the procedure described by Kennedy et al. (2004a, and 2004b). In the absence of contrary experimental or field evidence, allowable borehole pressures can be estimated where the plastic zone stretches halfway to the ground surface (providing between 20% and 30% reserve capacity). Parameters to use in these situations should be estimated by experienced geotechnical engineers. Preliminary values for cohesive force and the coefficient of lateral earth pressure at rest indicate that the blowout is the dominant failure mechanism in normally consolidated and lightly over-consolidated clays. Tensile fracture is expected in heavily over-consolidated clayey soil where the coefficient of lateral earth pressure exceeds 1.8 times.

While the solutions presented in this paper have been derived through careful consideration of the relevant soil behavior, they are theoretical in nature, and both field and laboratory studies would provide valuable guidance on the performance of this method.

Minimum Required Borehole Pressure from Cavity Expansion Method

The fluid pressure required to carry the cuttings to the surface is a critical factor in evaluating hydro-fracture risk. There must be a considerable difference between the minimum required pressure and the maximum allowable pressure to reduce the risk of hydro-fracture (Bennett et al. 2001). The minimum pressure primarily depends on the length, depth and the diameter of the borehole, the weight of the drilling fluid, and the flow rate. The minimum required pressure is a combination of the drilling fluid head pressure that must be overcome and the frictional resistance to flow between the fluid and the borehole wall. The following equation 3 is conservative and can be used to estimate the minimum required borehole pressure:

$$P_{min} = \frac{7.48\gamma_{mud} h_{bore}}{144} + L_{bore}\left[\frac{\mu_p v}{1000(d_{bore} - d_{pipe})^2} + \frac{\tau_y}{200(d_{bore} - d_{pipe})}\right] \quad [3]$$

Where,

γ_{mud} = unit weight of drilling fluid (lb/gal),

h_{bore} = height of mud column, or the difference in depth from a specific location in the bore to the surface of the entry pit (ft),

L_{bore} = distance from a specific location in the bore to entry point (ft),

μ_p = viscosity of the drilling fluid (cp) Soda Ash,

v = velocity of the drilling fluid = flow rate/area of bore annulus = Q/A (ft/sec),
Q = flow rate at the drill bit (gal/min),

d_{bore} = the diameter of bore hole (in),

d_{pipe} = the diameter of drill or product pipe (in),

τ_y = yield point of drilling fluid.

The Finite Element Analysis Model of Maximum Borehole Pressure through the ANSYS

The similar borehole conditions were simulated and analyzed using ANSYS cap model (Figure 5) to anticipate the borehole pressure and frac-out locations. The formulation of this model is described in Theory Reference for ANSYS and ANSYS Workbench from ANSYS documentation (Xia.H et al. 2006) and is briefly summarized.

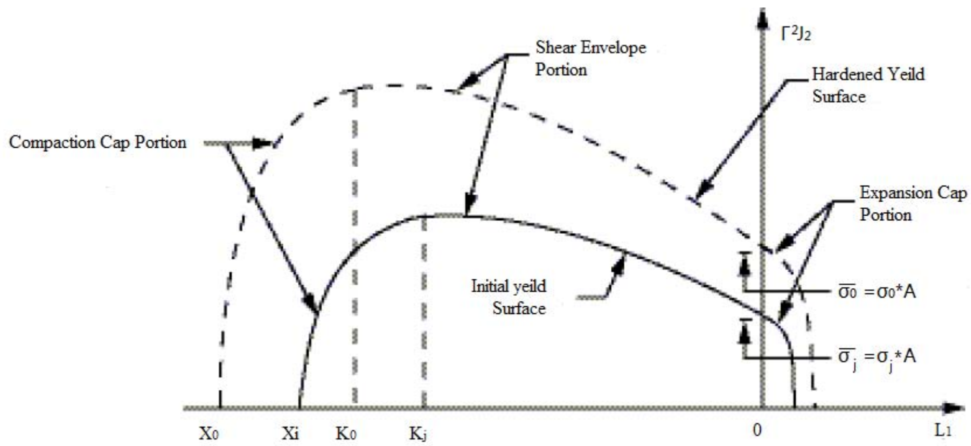

Figure 5. Cap model plasticity yield surface; tensile stress is positive

This model consists of a shear failure surface associated with an elliptical cap for compressive volumetric failure and a tensile cap, described by the following equation 4:

$$Y(\sigma,K_0,\sigma_0)= Y(I_1,J_2,J_3,K_0,\sigma_0)= \Gamma^2(\beta,\Psi)J_2- Y_c(I_1,K_0,\sigma_0)\ Yt(I_1,\sigma_0)\ Y_s^2(I_1,\sigma_0)\ [4]$$

Where,
Γ = Lode's angle function,
Y_c = compressive cap function,
Y_t = tensile cap function,
Y_s = shear failure surface.

This model was simulated to study the tensile behavior of soil using a cap plasticity model. The cap model data was collected from literature and parametric studies were conducted in horizontal boreholes to evaluate the conditions of volumetric failure. The study pointed that the axial stress concentration was the critical condition for volumetric failure around boreholes.

This prediction model for borehole stability analysis is based on soil mechanics model which was first proposed by Bradley (1979). Since then, several models based on continuous mechanics were developed. The assumptions of these models vary from simple elastic models to more elaborated elastic plastic models. Morita (2004) proposed an analytical procedure based on elasticity to evaluate the stress state around the borehole. The stress level is compared to a compression or tensile failure criteria to evaluate stability. The mostly used failure criteria are the Drucker-Prager, the Mohr-Coulomb, the modified Lade (Ewy, 1999) or the Hoek, and Brown criteria (Zhang and Zhu, 2007). A tensile criterion usually consists of the comparison of the minimum effective stress to the tensile force of the soil. The numerical modeling of borehole stability was analyzed considering the small strain, the small displacement, the classical associated plasticity theory, and the linear triangles mesh.

The borehole configuration was a horizontal borehole under plane strain conditions. To take the advantage of the problem symmetry, the mesh was consisted of 1/4 of the borehole geometry. The soil was treated as an isotropic and homogeneous continuum medium. The constitutive model used was the cap model as implemented by ANSYS. It also consists of a shear failure surface associated with an elliptical cap for compressive volumetric failure and a tensile cap (Figure 6).

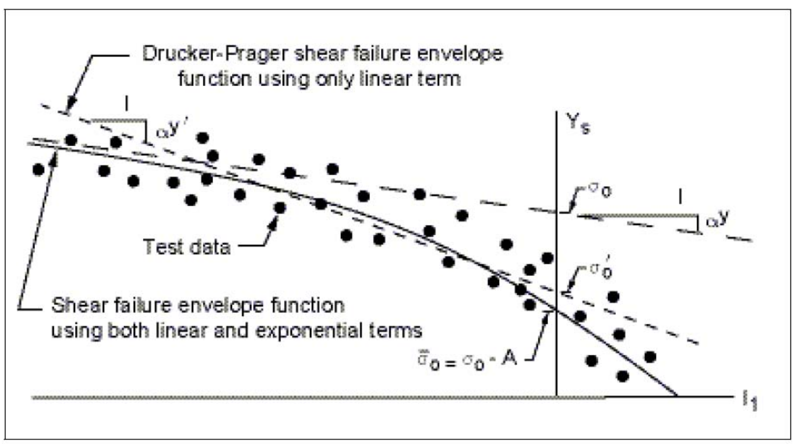

Figure 6. Shear yield function (from Release 14.0 Documentation for ANSYS)

2D-Basic ANSYS Model of the Horizontal Borehole

The borehole was represented by 1/4 of the circle to take the advantage of symmetry consisting of the horizontal external boundary of 32.8 ft., cover depth of 10 ft. from the borehole axis. The numerical model contains a total of 1,863 nodes and 880 elements. An example of mesh and nodal loading equivalent to the applied mud pressure is shown in Figure 7. The in-situ stress state was simulated by introducing a distributed vertical load on the top of the external boundary, whose value was equivalent to the vertical in-situ stress and a lateral load on the external boundary. The vertical effective in-situ stress was 7.25 psi and the horizontal effective in-situ stress was 1.45 psi. The in-situ stress in the axial direction was simulated by setting an initial stress state equivalent to its value. The pressure on the borehole wall is represented by pressure on the bore face. The Drucker-Prager materials model was picked up in this EFA, which is shown in Figure 8. The maximum pressure is 39 psi as 2D models predicted in Figure 9.

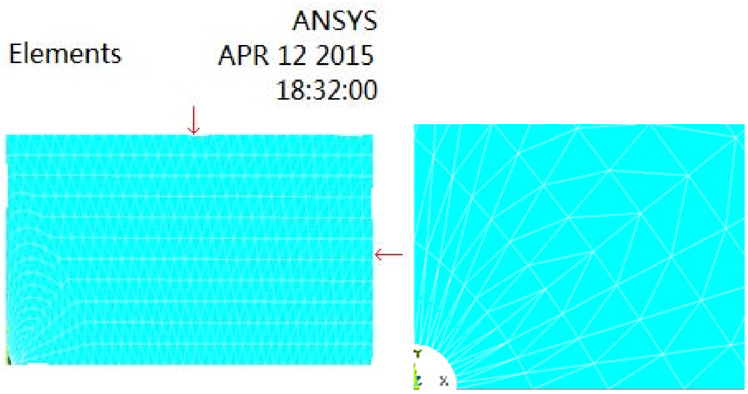

Figure 7. Example mesh and nodal load equivalent to applied mud pressure

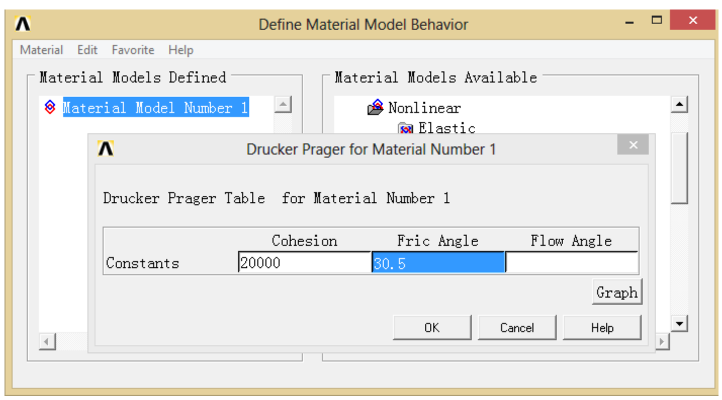

Figure 8. Parameters for 2D- materials modeling

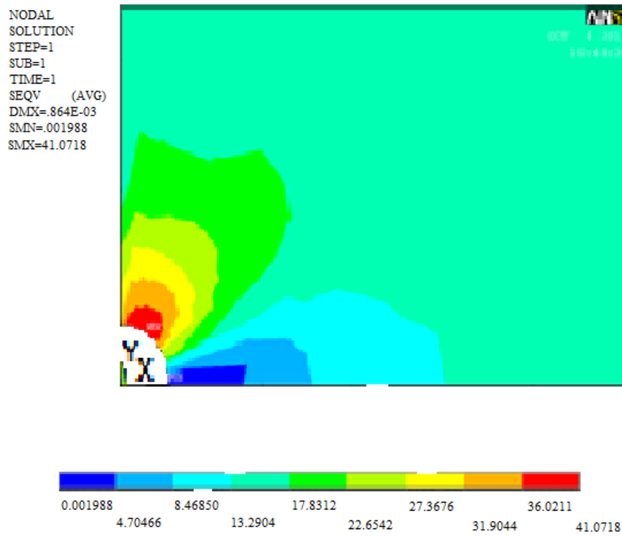

Figure 9. Primary stress and equivalent effective stress

3D-Basic ANSYS Model of the Horizontal Borehole

The new 3D numerical model was developed to simulate and evaluate the maximum borehole pressure during mini-HDD installation. The finite element analysis software ANSYS was employed. Plane 45 was chosen as the elemental material for 3D model. 2D numerical analysis (i.e. based on plane strain condition) was performed to examine the implications of assumption of plane strain conditions. The meshing used for 3D analysis and plane strain analysis is shown in Figure 10. The principal stress and the equivalent effective stress from 3D model were 39 psi and 41 psi, respectively (Figure 11). The parameters used for analysis by ANSYS are listed in Table 2.

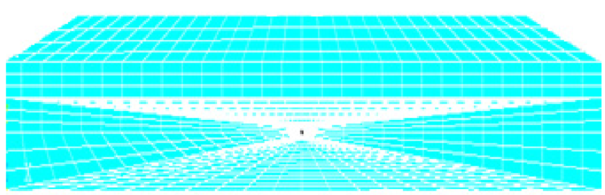

Figure 10. Meshes used for the three dimensional and plane strain analyses for 3D models

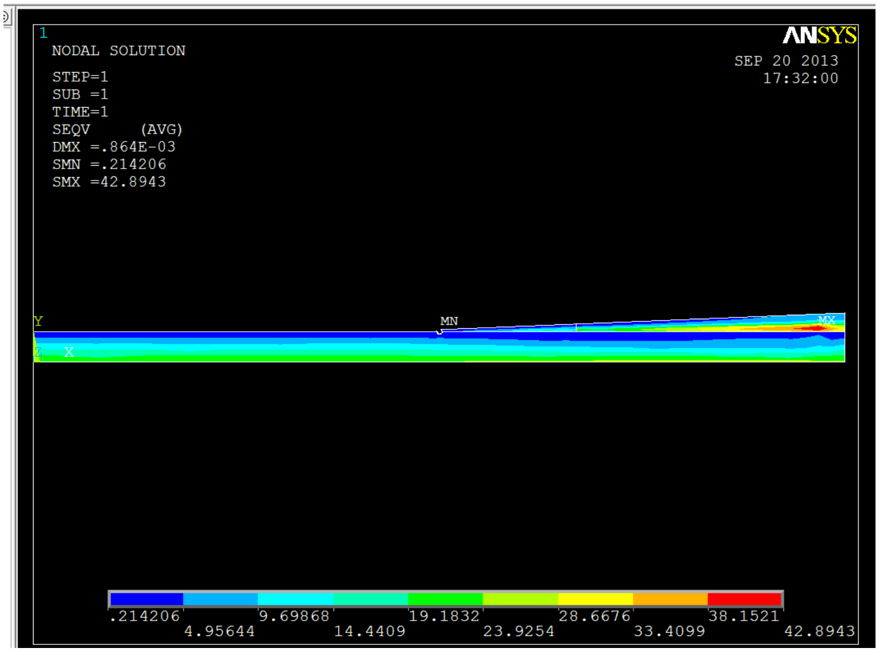

Figure 11. Principal stress and equivalent effective stress of 3D models

Table 2. The field data used in ANSYS

Borehole length, *ft*	260
Silty-clay height, *ft*	15.7
Clay height, *ft*	21.7
Fill height, *ft*	29.5
Depth, *ft*	10
Product pipe OD, *ft*	0.75
Product pipe thickness, *ft*	0.05
Product pipe ID, *ft*	0.702
Borehole diameter, *ft*	0.833
Mud layers diameter, *ft*	0.751

Conclusions

After plotting the maximum mud pressure from all three models in Figure 12 and table 3, it was revealed that the cavity expansion theory overestimates the allowable mud pressure, whereas ANSYS 2D and 3D models were conservative and predict closely to the actual result. Concluding on the above results, Drucker-Prager materials model which is adaptive for engineering application can be introduced in ANSYS as a new constitutive model library for better understanding and estimating of mud pressure.

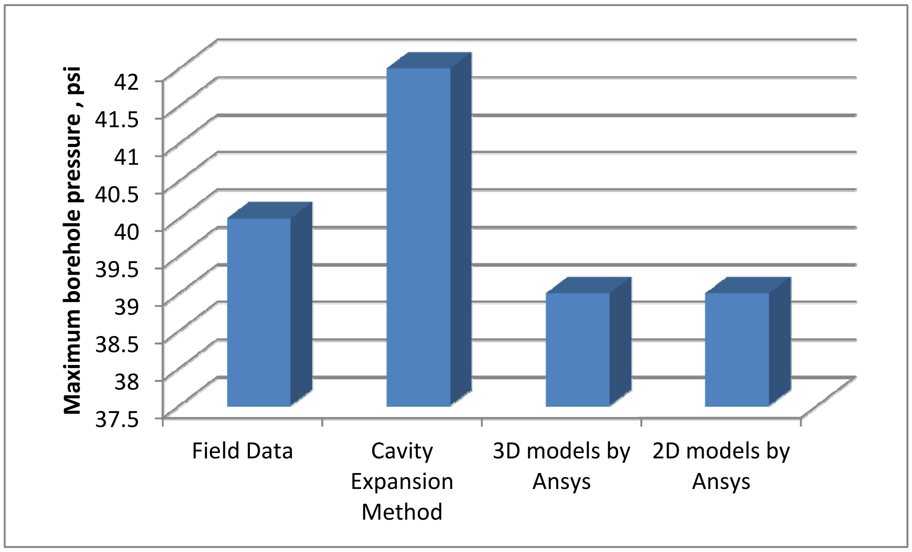

Figure 12. Comparison between maximum pressure (psi)

Table 3. Differences of Pressure

	Cavity expansion theory	ANSYS 2D and 3D models
Prediction Pressure (psi)	41.75	38.75
Differences of Pressure $\frac{(\lvert \text{Prediction Pressure} - \text{Field Date} \rvert)}{\text{Field Data (39.75 psi)}}$	5%	2.5%

The modeling procedure used was able to capture the volumetric compressive behavior of the soil around the borehole. Cap models present different formulations. To use them, it is necessary to work directly on experimental data, once different formulations lead to different parameters. This is the major drawback of this model.

Elastic-plastic cap plasticity models are able to define a damaged zone. The size of the plastic zone that induces borehole instability is the other topic of research. In the field, this mechanism is not completely understood. It is known that plastic compaction disaggregates the material, causing bonding and grain breakage. Whether this material will be carried by fluid flow or it will produce a compacted region that acts as a barrier to flux should be investigated by more refined coupled models. Localization models, multi-scale models or FEM-DEM coupling would be helpful in understanding the role of compaction in borehole instability.

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Victory Pipeline Duchesne County Utah Water Conservancy District

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Abstract

The Victory Pipeline project is a $30 million dollar, 27.5 mile pipeline located in Duchesne County Utah. This gravity fed pipeline was constructed using HDPE pipe with sizes ranging from 22”-30” that will deliver treated potable water to 7 different water companies and cities. This paper will discuss the pipe material selection process, pipe fusion, and installation techniques including open trench, and HDD and the benefits of using HDPE pipe. Several obstacles were encountered in the design and installation of this project which forced an alignment change. These obstacles include environmental assessments, endangered plant species, migratory birds, easements of over 50 property owners, state land, federal land, the Bureau of Indian Affairs property easements, and coordination with US Fish and Wildlife, and the US Bureau of Reclamation.

PURPOSE AND NEED

Duchesne County, Utah encompasses approximately 3,250 square miles in the eastern portion of Utah. The County has two larger population centers with the towns of Roosevelt and Duchesne. The remaining portion of the county consists of agricultural lands, small rural communities, tribal lands, and is the largest crude oil production county in Utah.

Figure 1 - Duchesne County

The municipalities of Roosevelt and Duchesne have developed water systems to serve these two larger population centers. To meet the needs of the smaller rural community, agricultural, and energy production demands, several smaller water companies have evolved. The development of these water companies has resulted in several smaller sized water systems running throughout the County to meet immediate growth demands, and in some cases, overlapping or sharing common facilities. This has resulted in several utilities being constructed in public rights-of-ways causing significant congestion.

Additionally, all water suppliers in the area are heavily impacted by the rapidly growing petroleum industry. This additional industrial water demand creates a heavy strain to most of the local water suppliers.

Duchesne County Water Conservancy District (DCWCD) was formed in 1998 under State Code Title 17B and is a legal subdivision of Duchesne County, UT. The mission statement of the District, in part, is to construct, operate and maintain facilities associated with water resources and such other facilities as are necessary to the functioning of the District. In 2010, as a step towards this goal and to meet growing demands, the Central Utah Water Conservancy District (CUWCD) agreed to expand their Duchesne Valley Water Treatment Plant (DVWTP) capacity from 4 million gallons per day (MGD) to 8 MGD. The treatment plant source water is surface water from Starvation Reservoir. As part of this agreement, DCWCD agreed to construct a pipeline that could deliver the finished water from the treatment plant to customers throughout a major portion of the County.

Planning for a pipeline project to deliver this water began in 2008 and has developed over several years. To accomplish objectives that were developed throughout the planning phase, several project goals were established including:

- Utilize the 4 MGD made available by the CUWCD DVWTP expansion in 2010.
- Provide a reliable source and transmission system for several participating customers.
- Provide a shared customer water system that reduces expense by eliminating the need for more costly independent systems.
- Improve existing customer system hydraulic capacity by injecting high pressure source flows at critical and bottleneck points in existing customer delivery systems.
- Provide a reliable source for customers currently dependent on surplus water.
- Provide a secondary water source for several customers.

BASIS OF DESIGN

In 2012 a formal alignment study and planning level cost opinions were completed. The study analyzed several alignment alternatives considering land ownership, hydraulics, geology, and other factors in constructing a pipeline that could deliver finished water to seven existing water companies, districts, and municipalities (customers). These customers serve 90 percent of the County’s population. The project was subsequently named the Duchesne County Victory Pipeline (DCVP).

Following the alignment study a cost/affordability analysis was conducted considering customer affordability criteria for their retail water sales. The results of the study concluded that the project could not exceed the planning level cost opinion of approximately $34 million or retail water rates would need to be raised beyond what customers would be willing to pay.

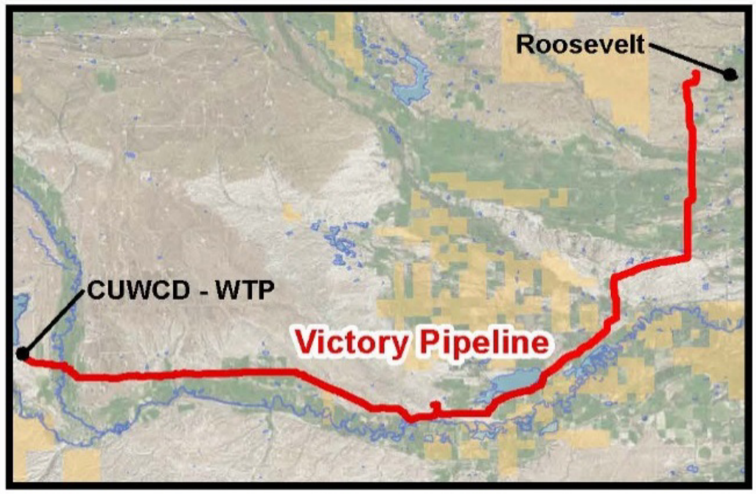

Figure 2 - Victory Pipeline

The DCVP traverses 27.5 miles of undulating ground through foothills, agricultural lands, rock formations, wetlands, environmentally sensitive lands, roadway rights-of-ways, multiple canal crossing, stream crossings, and a river crossing. The alignment avoids urban areas and other population centers. Landowners include the United States of America (US Bureau of Reclamation), State road crossings, Federal grounds (US Bureau of Indian Affairs), and private property.

Planned customers have shown over the past years that they have adequate resources to operate and manage their individual delivery systems, but some of these systems are at capacity. The DCVP delivers up to 4 MGD a day to vastly improve customer systems by providing a reliable source of water at strategic points in their individual delivery systems.

The topography generally has a declining slope from the CUWCD treatment plant to the project termination in Roosevelt. This lends itself to a complete gravity system with customer turnouts strategically located to take advantage of improved pressures.

In the course of design, a geotechnical report was completed that identified soil types of poorly graded gravel, sands, silt sand, sandy clay, and lean clay with shallow ground water in two locations. Perhaps the most important discovery from the soils analysis concerns the corrosion potential for ferrous metal. The resistivity analysis indicated that the onsite soils have resistivity values ranging from 160 to 3800 OHM-cm and pH values of 7.30 to 8.86. Based on these results, the onsite native soil is expected to be very corrosive to moderately corrosive (Mattson, 2014).

Since DCWCD has limited maintenance staff, maintenance and operation became a primary consideration during the design. With this in mind, the following objectives were developed:

- Construct a complete gravity system from beginning to end.
- Minimize energy costs.
- Reduce potential for pipeline leakage.
- Pipeline materials with little to no corrosion potential.

- Limit materials and equipment required to operate the pipeline.
- Mechanical equipment with low maintenance requirements.
- System longevity.
- Minimize impacts to private land owners.
- Best product possible with fixed funding.

HYDAULICS AND PIPE MATERIALS

To meet the objectives of the project, several material types were considered for both the pipeline and required mechanical equipment (valves, meters, etc.). These materials were evaluated for hydraulics, capital costs, life cycle costs, maintenance and replacement requirements, and ease of operation.

Because of the undulating topography, the pipeline hydraulics was carefully evaluated using conservative values where possible. Determination of the pipeline hydraulics used the Hazen-Williams equation in a spreadsheet analysis:

$$hf = \left[(10.44)(L)(\dot{V}_{gpm})^{1.85}\right] \div [(C_{HW})^{1.85}(d_{inches})^{4.8655}].$$

For plastic pipe a conservative Hazen-Williams coefficient (C_{HW}) of 140 was used. For concrete lined steel and ductile iron pipe, a conservative C_{HW} value of 100 was used.

The design flow of 5 MGD (peak day) with a maximum velocity of 5 feet per second were used as criteria in determining the required pipe size. Using these criteria, internal pipe diameters were evaluated ranging from 18 inches to 30 inches. The results showed that a combination of pipe diameter consisting of 24 and 18 inches, for a C_{HW} value of 140 were required to meet the project hydraulic conditions, and pipe diameters of 30 and 20 inches, for a C_{HW} value of 100 were required to meet the project hydraulic conditions

In evaluating pressures in 18 and 24 inch pipelines for both C_{HW} values , there was an approximate pressure differential of 65 psi (equivalent to 150 feet of head differential) between the two over the pipeline length of 27 miles. More importantly, there were three locations along the pipeline alignment where pressures were near or below 0 psi when using a C_{HW} of 100.

In addition to low pressure areas along the pipeline alignment, the opposite was also true. There were areas of higher pressures. Since this is a gravity system, static pressures were analyzed for the system high pressure requirements. At the lowest elevation point in the system the high pressure was approximately 270 psi.

The results of the hydraulic analysis demonstrated that the pipeline needed to possess the ability to accommodate a wide range of pressure conditions. Given the results of the hydraulic analysis and the corrosion potential, plastic pipe materials became the preferred option.

The two most common types of plastic pipe materials for water systems, High Density Polyethylene (HDPE) pipe and Polyvinyl Chloride (PVC) pipe, were then carried forward in the pipe material selection phase of the project.

HDPE pipe offers a welded joint pipe with pressure ratings ranging from 63 psi to 333 psi for PE 4710 cell classification. PVC offers standard pressure ratings ranging from 80 psi to 235 psi for 24 inch Fusible C905®. PVC also offers a fusible joint product.

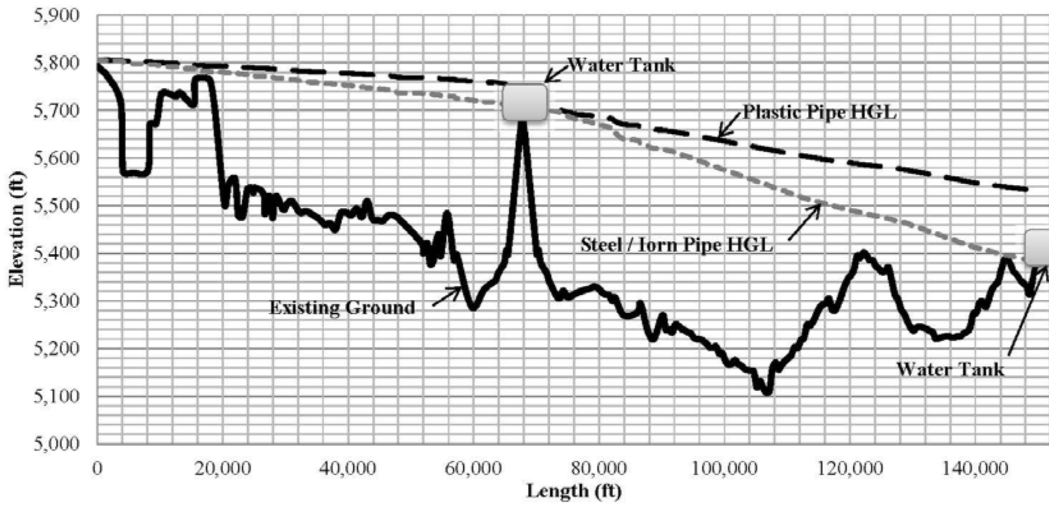

Figure 3 - Hydraulic Analysis

To satisfy the objectives of potential pipeline leakage, system longevity, reduce installation time through private/agriculture lands, and accommodate installation along undulating terrain, a welded joint pipe was desired. A selection of welded joint pipe reduces the number of fittings along the alignment as well as eliminates the potential for joint deflection and potential leakage inherent in bell and spigot joints. HDPE pipe was chosen due to its ductility and tighter bend radius compared to other fusible pipe materials. The HDPE fusion process has an approved ASTM F2620 with historical performance data that produces a pipe fusion that is leak free and is as strong as the pipe itself (ASTM F2620).

Although a plastic pipe material became the preferred alternative for this project as the main piping material, steel pipe was used inside most of the customer turnout vault structures and major drain/isolation vaults, isolating the steel pipe from the corrosive soils. These vault structures became ideal locations to include restraint for HDPE pipe expansion and contraction due to temperature fluctuation. Bolted flange adapters were used to transition from HDPE pipe to steel pipe inside the vault.

COST ANALYSIS

Since metallic pipes didn't satisfy a number of the Victory Pipeline project objectives, they were not included in the cost analysis. Comparing fusible HDPE pipe with internal diameters matching that of Fusible C905® PVC showed that the difference in costs was negligible. The difference in material costs were offset by the lower installation costs of HDPE pipe.

BENEFITS OF HDPE PIPE

HDPE pipe is becoming more commonly used in municipal and irrigation water projects across the country. The findings of the DCVP are becoming apparent to the industry as more people are going away from traditional pipe materials that have documented historical problems and reaching out to HDPE pipe as the solution. Many engineers/designers are either first introduced to HDPE pipe through trenchless technologies where HDPE pipe is the leading material used for horizontal directional drilling, pipe bursting, and other trenchless technologies due to its ductility, strength, abrasion resistance and toughness. Or they are introduced to it through their gas distribution counterparts where polyethylene pipe has been used for decades of leak free performance.

There are many construction/installation advantages to HDPE pipe, such as narrower trench widths. HDPE pipe is fusion joined above ground and then placed into the trench. This eliminates the need to over excavate for trench boxes in order to put people down in the trench safely. This reduced excavation greatly saves time and money, allowing HDPE pipe to be installed quickly and efficiently Figure 4.).

Figure 4. HDPE Pipe Installation

HDPE pipe has the tightest bend radius of any pipe material, 100 times tighter than other fusible plastic pipe products. Where direction changes need to be performed in a small area, fabricated HDPE fittings are available. These can be pre-ordered or fabricated on site (Figure 5.). Fabricated fittings are fully heat fused and do not require thrust blocks at directional changes like other unrestrained joint piping materials. This saves time and cost during installation.

Figure 5. Fabricated Fittings

HDPE pipe has decades of documented history on fusion performance. Butt fusion procedures ASTM F2620, coupled with the Plastics Pipe Institute's (PPI) TR33 are the only plastic pipe fusion procedures to be adopted by the American Society of Mechanical Engineers (PPI TR33). HDPE fused joints are fully restrained and leak free. Rural Utah is known as the king of canals, many of these canals were built when the early pioneers originally settled Utah. These canals have served the area well for over 100 years, but with populations growing and water becoming more scarce in the drought stricken Western United States, water conservancy districts across the country are opting to close in these canals using HDPE pipe. These original open canals have a historical average of 30% water loss due to evaporation and seepage. Other bell and spigot piping materials have been used in the past, but losing water through exfiltration or contaminating good water through infiltration, leak free HDPE pipe is now being used to control these problems.

Surge resistance is an important part of any pipe design. HDPE pipe is one of the most surge resistant piping material and has the ability to withstand occasional surges up to 100% above the pressure rating of the pipe and recurring surges at 50% above the pressure rating of the pipe (Figure 6.). This ability comes from the ductility of the material coupled with its ability to expand during water hammer events. Chapter 6, Section 1 of PPI's Handbook of PE Pipe can be used to aid in design of HDPE pipe systems. This section includes design criteria for surge allowance (PPI, Ch.6 2008).

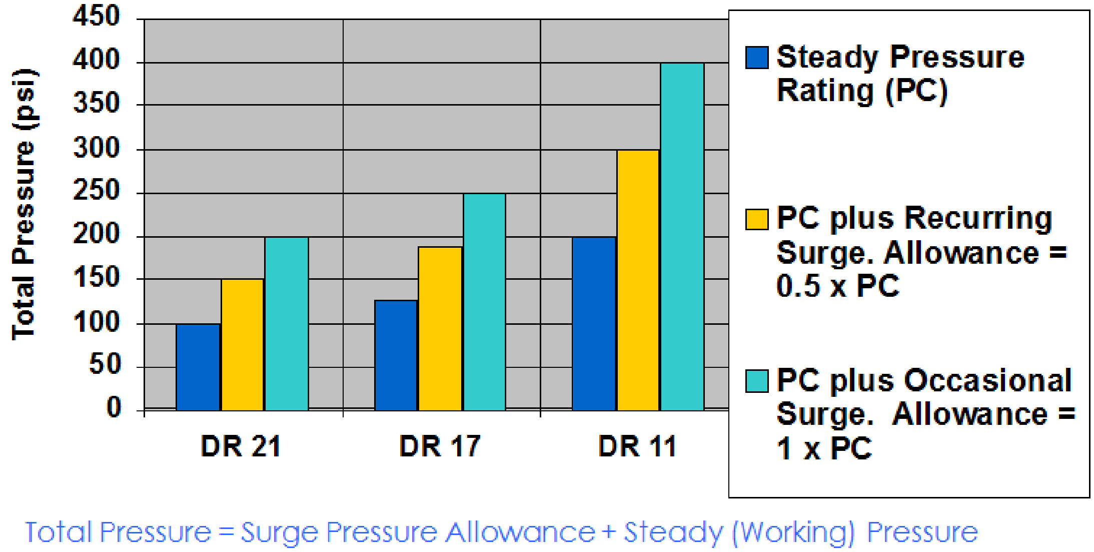

Figure 6. HDPE Pressure Surge Allowance Chart

Further surge design assistance can be found on PPI's website using the PACE calculator found at http://ppipace.com/ (Figure 7.).

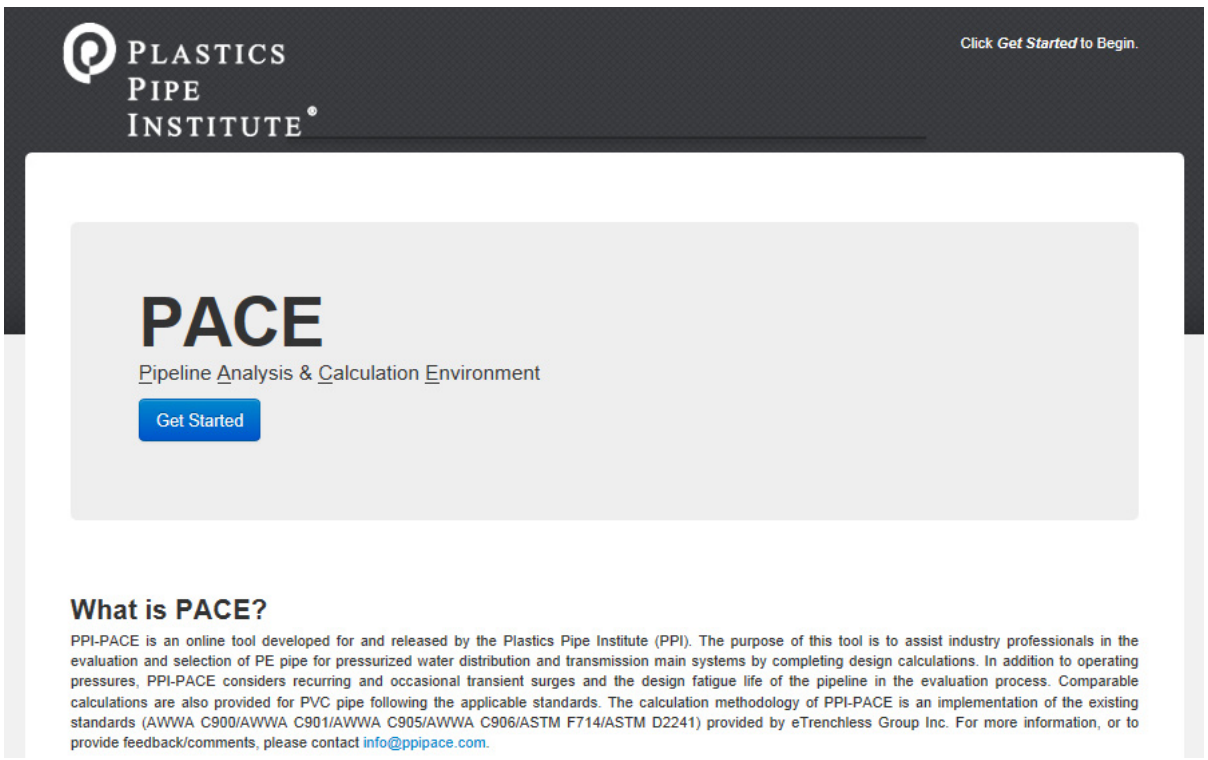

Figure 7. PPI's PACE Calculator for Pressure Surge

HDPE pipe is resistant to corrosion and chemical attacks and can easily handle corrosive soils with no degradation. HDPE pipe is resistant to biological attack and does not support the growth of tuberculation or build up on the inside wall of the pipe. This allows HDPE pipe to maintain its original Hazen Williams C factor of 150 for the life of the pipe (PPI, Ch.6 2008). These benefits combined with low maintenance requirements give HDPE pipe the lowest life cycle cost of any pressure piping material with an estimated life span greater than 100 years.

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Big Pipe—Tight Quarters: Lessons Learned from Large Diameter Urban Pipelines

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Abstract

Large diameter pipelines are a critical component of a utility's infrastructure. Design and construction of pipelines in an urban environment is a difficult task. Weaving a large, critical piece of infrastructure through city streets, easements, and existing infrastructure has unique challenges that must be studied in the planning, routing and preliminary design of the project. This paper will present an outline for the planning, design and construction of a large diameter pipeline in an urban area based on lessons learned from previous projects. The lessons learned during construction provide valuable insight into how better planning and design can greatly reduce the challenges faced in future construction projects. Pipeline construction can impact businesses, bring traffic to a halt and jeopardize public safety. To limit these impacts to the community, engineers must consider public involvement initiatives, traffic control and project phasing. Route selection is a critical part of the process and can ultimately have the greatest influence on how challenging construction and maintenance of the pipeline will be. Additionally, engineers can work within the framework of the triple bottom line to balance, environmental, economic and social impacts. This process can be used in evaluating open cut construction versus tunneled construction as well as selecting the best route for the pipeline. Construction of large diameter pipelines is slow and expensive. Working with contractors to identify work space requirements and construction methods is very important. As a part of this paper, several contractors were contacted to discuss these challenges and a summary of those discussions is included in the paper. Pipelines, due to the large investment and criticality of service, must be reliable and resilient with a long service life; therefore, they must be engineered with robust design criteria and with operations and maintenance in mind. This effects route selection, pipe material selection, backfill and embedment design and appurtenance layout. The earlier these factors are considered the better off the project will be during construction.

INTRODUCTION

Large diameter pipeline design and construction in an urban environment is a difficult task. Some of the challenges that a designer and contractor face include working in tight spaces, dealing with utility conflicts, minimizing impacts to the public and risks associated with prolonged schedules and escalating costs. Similarly, pipeline owners face challenges in the operation and maintenance of the pipeline including leaks or blow-outs, damage from third parties and difficulty accessing the line in tight spaces with new development.

The purpose of this paper is to provide lessons learned and best practices for dealing with these challenges. The focus of the design engineer should be on providing reliability and flexibility over the long term while balancing the ability to construct the pipeline. This can be accomplished with a thorough route selection process, utilizing robust standards during final design, and considering the future use of the pipeline to make the pipeline easier to operate and maintain.

ROUTE SELECTION

A thorough route selection process can reduce cost of construction, operation, easement acquisition, environmental impacts, impact on landowners and schedule (Hutson, 2006). The schedule savings can be in terms of reducing permitting requirements, shortening the design process and reducing construction time.

Data Collection. Better decisions are made with better data. Engineers must take the time to gather the appropriate data to improve the decision making process. Geographic Information Systems (GIS) provides a perfect platform to collect and utilize the data. Recent aerial photography of the pipeline corridor will provide a great base for the data. Aerial data can be acquired from mapping services such as ESRI, USDA, USGS, Bing, Google, and many others. Owners often have access to recent aerial data as well. Many municipalities have invested in asset management systems for their water and sewer utility systems as well as their other infrastructure. This data is invaluable in determining utility conflicts. Other good data sources include:

- County Appraisal Districts for property lines and landowners
- The railroad commission for buried oil and gas infrastructure (Texas only).
- FEMA for floodplain boundaries
- EPA and Historical Commission for environmental and archeological data
- USDA for soil data
- USGS for land cover and ecological data
- National Wetlands Inventory
- ESRI for general U.S. and World mapping information
- Utility master plans

Engineers should also research future development plans in the form of zoning maps, comprehensive plans, master thoroughfare plans, and capital improvement plans.

Alternatives Analysis. This may be the most critical stage of the route selection process. The development of alternatives is a brainstorming process in which no idea is ruled out immediately. Multiple routes, corridors, in streets versus in easements, and installation methods (tunnels versus open cut) should be considered and then screened out using selection criteria. Selection criteria may include the following:

- Initial capital cost
- Life cycle cost including capital, operations, and maintenance costs
- Constructability
- Impacts on schedule
- Vulnerability to 3rd party damage, soil erosion and future development
- Accessibility to perform O&M
- Environmental impact
- Social impact (road closures, lost business, traffic delays, etc.)
- Easement acquisition cost and schedule

The shortest route is not necessarily the best, all factors need to be weighted and assessed to determine the best route for the pipeline. The initial capital costs should be balanced versus life cycle costs and short term and long term impacts on the environment, businesses and community must also be considered. This evaluation process is known as the triple bottom line (TBL). The TBL is an accounting framework with three parts: social, environmental and financial. Utilizing this framework will allow the engineer to make decisions throughout the project life that take into consideration all the stakeholders affected by the project.

Installation Methods. There are two options for the installation of large diameter pipelines in urban areas. Open cut will usually have the least cost; however, in an urban environment, some tunneling is usually required to cross obstacles such as highways, railroads, rivers and, other large utilities. The route selection should evaluate how much tunneling will be required versus open cut.

At times, long, deep tunnels can be used to improve hydraulics of the system and reduce power usage by reducing system high points. Other times, tunnels are built out of necessity due to limited space for open cut, substantial critical underground infrastructure, or other severe impacts to the public.

Easement Needs. The workspace requirements to construct and maintain the project need to be considered during the route selection phase. In some instances, permanent and/or temporary easements may be required. At other times, the contractor may work in the public right-of-way without requiring easements. Regardless, the following considerations must be made:

- There must be enough room for the contractor to string pipe and embedment materials, move his equipment around for trenching and backfilling as well as room for temporary spoil.
- Remote laydown areas may be needed to string pipe and other materials; however, double handling of pipe and materials results in added cost.
- Access easements may be required where public road access is not available.
- Tunnel pits may require additional temporary workspace and staging area.
- HDD installations have additional site requirements that must be met, currently HDD is limited to about 48-inch diameter pipe
    - Depending on the diameter, each side of the HDD may require as much as a 100’x150’ laydown area.
    - Additional easement may be required to string out the pipe so it can be fused and laid out before it is pulled.
    - The maximum bending radius of the pipe may limit where the pipe can be strung out prior to the pull.
- Room is also needed for the owner to properly provide routine maintenance, repairs to pipe and pipe joints, valves and to make tie-ins.
- Purchase exclusive easements to prevent encroachment from future utilities.
- In some deep tunnel applications, subterranean easements are needed that vary significantly from open cut easements.
- Major appurtenances may require additional easement

Access. It is important to consider how the pipeline site can be accessed both during construction and after it. The contractor will need to be able to access the site, store equipment, and move material, pipe, and appurtenances in and out from the site while the owner will need to be able to perform maintenance or make any repairs that are needed. Many access points are needed for to allow this to happen without negatively impacting production or operations and maintenance.

Traffic control plans are not just important for minimizing traffic delays but also for allowing the contractor to easily access the construction site. Additionally, if a pipeline is to be placed under pavement it is preferable to locate the line in an exterior lane so that only one lane must be shut down in the future for construction and maintenance access.

Subsurface Utility Engineering (SUE). SUE is an invaluable tool for pipeline route selection and design. There are several different quality levels that each provide differing amounts of detail as to the location and/or depth of existing infrastructure. Quality levels are defined in ASCE Standard 38-02. Quality Level D and C can be valuable for route studies. Level D SUE uses data from existing utility records while Level C locates visible facilities such as manholes, valves boxes and pipeline markers to correlate the Level D data. Quality Level B data involves geophysical methods to determine the horizontal location of all underground utilities, while Level A involves potholing utilities to verify the exact horizontal and vertical location. In some instances it may be important to utilize Level B SUE data before completing the final route selection and alignment determination. Level A should be completed before starting preliminary design.

In numerous instances, engineers have relied on as-built data on waterlines and other utilities to set tunnel depths only to find that the depth of the utility is much deeper than it was shown on the as-builts. In other cases, utility lines that were assumed to be properly centered in their easement were actually installed well outside the easement. When paralleling existing easements, SUE Level A should be used to verify the location of the adjacent utility.

DESIGN

Several elements of the pipeline design are critical to the long term performance of the piping system. This section will focus on these key elements and design tips to achieve long, reliable service life with minimal maintenance.

Pipe Material Selection. There are a handful of pipe materials suitable for large diameter transmission pipelines. These options include Steel Pipe (AWWA C200), Pre-stressed Concrete Cylinder Pipe (AWWA C301), and up to certain diameters: Ductile Iron (AWWA C151), Bar-Wrapped Concrete Cylinder Pipe (AWWA C303), Polyethylene Pressure Pipe (AWWA C906) and Fiberglass (AWWA C950).

All projects are different and various pipe materials may be suitable for some projects and not for others. In urban environments the mode of failure is also critical as catastrophic failure can put lives and property in danger.

Embedment and Backfill Design. In an urban environment, factors that influence the embedment and backfill design include the long-term reliability of the trench system, reduction of settlement especially under pavement, the ability to place materials and backfill quickly, and even the protection of the pipe from third party damage. In some situations, flowable fill can provide superior support, reduce settlement, allow backfilling to proceed in less than an hour, and provide some measure of protection from third party damage. Flowable fill is preferable over lean concrete due to its ability to be excavated without jackhammers. The Engineer and Owner must weigh the additional cost of this embedment system versus the short term and long term benefits.

Large diameter pipelines can create large obstructions for other utilities if sufficient cover is not provided to allow those future utilities to be placed above the pipeline, especially gravity lines for stormwater and sanitary sewer. However, extra depth translates to extra cost so the engineer can’t be so conservative that prices are driven up unnecessarily. Significant effort should be given to coordinating with any future development to determine the correct depth of cover to place the pipeline.

Tunnel Design. Quality geotechnical information is critical to the design of tunnels for large diameter pipelines. The risks associated with tunneling can only be assessed once quality data is provided to the Engineer. Factors such as soil classification, groundwater depth, and other soil strength characteristics must all be evaluated to determine if tunneling is the correct method of construction for a given location.

If groundwater levels are very high, significant dewatering measures may be required and potentially the use of tunnel boring machines (TBM) to construct the tunnel.

Appurtenances. Special consideration must be given to pipeline appurtenances in an urban environment. Engineers should plan for future tie-ins and include fittings to assist with hydrostatic testing and disinfection. Coordination with the Owner and project stakeholders is crucial to plan for future connections.

A transmission pipeline in an urban setting may require more main line valves to enhance the ability to maintain the pipe by reducing the amount of dewatering that is needed if man access is required and can isolate line breaks or sections for repair.

Pipeline construction in an urban environment can be combined with paving replacements to reduce total costs. In the case of concrete paving, the pipeline can be positioned where a panel replacement is satisfactory. When the street is in poor condition, a full replacement may be warranted. Asphalt paving can be upgraded with a mill and overlay.

Cathodic protection is a must for long term protection of all ferrous pipe materials. This is particularly true of large diameter pipelines that require a high initial investment that must be protected. Large pipelines in an urban environment are even more critical due to the cost to rehab/replace and the consequence of pipe failure. To make cathodic protection systems more robust, plan for redundant connections to the pipe with short runs and wire protected in conduit. Isolation of pipelines is critical to make the cathodic protection system efficient. Corrosion engineers must address interference with other piping systems and adjacent cathodic protection systems. Determining a source of power is also a critical planning component and locating the rectifier for an impressed current system such that it can be easily accessed and maintained is an important step in the design.

Constructability. Constructability is one of the larger challenges associated with urban construction. What may come easily in a rural pipeline project can be much more complex and difficult in an urban pipeline project. Caution must be taken when excavating near so many existing facilities and that causes construction to take much longer. Many utilities and pipeline owners require that excavation near their facilities be done by hand which can significantly slow down construction progress.

Construction equipment can also be limited in the head space available due to structures or overhead power lines. The equipment may also be limited in its mobility horizontally due to narrow easements or tight working conditions. Additional construction equipment may be required on site if some equipment is not able to freely move around. Site conditions may also require the use of rubber tires instead of tracks to prevent damage to pavement. These are all limitations placed on the contractor which can affect production rates and costs.

Haul routes may also need to be considered to establish how material can be brought to or removed from the construction site as needed. If haul routes require longer trip lengths for trucks it may increase costs and slow production. Counties and municipalities may limit the timing for these operations to off peak times.

In urban areas there is likely a need for continuous access to businesses, residences, and offices that may create many subdivided sections in the construction. Traffic control plans need to provide access to these facilities and the engineer must consider how much space there is between access points to ensure it is adequate for the installation method specified.

All of these constructability issues need to be taken into consideration during design. A good alignment must take into consideration the final location of the pipeline as well as the ability to install the pipeline in that location.

Cost Estimating Considerations. Preparing an accurate opinion of probable construction costs for an urban pipeline is difficult. Some tips that can increase the accuracy of the estimate are as follows:

- Collect data on similar urban pipeline projects to help develop not only unit costs but some big picture perspectives on total project costs.
- Be careful not to use past project bid tabs exclusively. All projects are different and have different bidding environments and markets.
- Material costs are typically similar for urban projects; however, slow construction in a congested environment may increase installation cost.
- Restoration of paving and landscaping is expensive but can be quantified.
- Talk to contractors to get their feedback on crew sizes, production rates, special equipment needs and other factors that influence cost.
- Keep up with pipe and construction market conditions.
- Consider the rate of production in the cost, the longer it takes to install the pipe the more it will cost.
- Account for contractor's risk associated with working near so many existing facilities, this risk will be reflected in a higher bid price.
- Traffic control may require concrete barriers and flagmen.

It is difficult to quantify how much additional cost may be incurred by the slowed production rate of construction in urban areas which is why it is important to discuss the project early on in design with contractors to remove as many limitations on the construction as possible.

CONSTRUCTION

An Engineer's work does not end with design. Construction in urban areas requires significant oversight and coordination between all parties involved in the process from planning through construction.

Risk Assignment. An important factor to consider when designing and planning for a pipeline project in an urban area is the risk involved in construction.

Working in an urban environment brings additional safety risks that must be considered, such as working near traffic, proximity to pedestrians, hazardous underground utilities, and potentially contaminated soils. Proper barriers or construction fencing is essential for public safety as well as the safety of the construction workers. Thorough geotechnical work can help mitigate some risk.

As previously discussed, SUE can help to clearly identify the existing infrastructure near the project. Leaving the locating up to the contractor during construction can lead to significant design changes, lost time, and increased costs when unforeseen utilities are encountered. If some utilities cannot be located until construction, require that the utility be located at the beginning of construction so that any changes can be made without negatively impacting the schedule.

The project specifications need to clearly address how conflicts are handled and lay out a process to resolve any issues as quickly as possible so the project schedule is not significantly impacted. Additionally, the Owner may consider adding an allowance or contingency in the bid to allow for unforeseen issues.

Another way to mitigate risk it to collect survey information for adjacent structures and facilities that are near the construction site to confirm if there has been any negative impact. Taking good pre-construction photos can also help with this. Tunneling operations are particularly important to monitor due to the potential for settlement.

Public Involvement/Communication. As a part of the social considerations in the TBL, pre-construction meetings can be hosted to allow the public to ask questions about the project and understand how the project may impact them. These meetings can also help the project team to understand the concerns of the community so that they can be addressed.

It is also important to keep the public informed on construction progress throughout the life of the project. This can be done through mailings to landowners, door hangers for homeowners, or a public website that is regularly updated with construction status reports. Above all those things though, one on one communication can do the most good in mitigating any issues that may come up with the community.

Scheduling. Construction scheduling is an important consideration for the success of the project. Various seasons and events should be considered when laying out the construction schedule. Some of the factors that may require consideration are:

- School year, consider constructing near schools during summer months.
- Peak shopping months.
- Holidays.
- Community events (parades, fun runs, sporting events, etc.).
- Wet weather, allow time in the construction duration for rain delays.

If service connections or other pipeline connections are required, consider the timing of these connections. Some may require night or evening work to reduce service outage impacts.

Construction Data Collection. Since large diameter pipelines are such critical infrastructure, it is important that accurate data be collected and stored for future use in repairs or maintenance. It is also important to properly identify joints and specials as well as keep up-to-date records of the pipe as it is being installed. Accurate record data should be maintained throughout the project.

Survey data is also important to maintain accurate data on elevation and alignment of the pipeline to prevent third party damage in the future and allow maintenance crews to easily find and maintain the system.

Construction Inspection. Generally, construction in urban areas will require more oversight than in less developed areas. A good construction manager can save the owner significant amounts of money by addressing construction issues before they become major problems. The construction manager can also act as the owner's representative to the community to help address concerns and issues that come up during construction.

Materials testing is also an important part to construction. As previously mentioned, the embedment design is critical to the strength and long term performance of the pipe and ensuring that the contractor uses the proper materials and meets the installation requirements from the design is essential to successful construction of the pipeline.

INPUT FROM CONTRACTORS

As a part of this paper, several contractors were contacted to discuss the challenges faced and important considerations for designing and constructing large diameter pipelines in urban areas. The following points reflect some of the input provided by experienced contractors in this field.

Trucking. The most common challenge voiced by the contractors was the difficulties involved with bringing in and removing construction material. Large pipelines can potentially require huge amounts of embedment and backfill material to be imported to the job site, the contractor's ability to do this greatly impacts the cost and schedule of the project. It is critical to ensure adequate access to the construction area during allowable working hours. To help address this issue one may consider using native material or some combination of native and imported material to reduce the amount of imported material. On-site recycling of asphalt and concrete was also suggested as an alternative. Removing material also creates a large demand for trucking. A good rule of thumb is that the maximum amount of material that can be removed in a given day is 1,000 CY.

Traffic Control Planning. Early communication and coordination with the public to mitigate traffic concerns can help to reduce scheduling issues and public impact. If it is possible to implement detours and/or complete closures at street and intersection crossings in lieu of constructing the intersection half at a time it will speed up construction and reduce the overall time that traffic is impacted.

Working Room. A lack of working room can severely limit the contractor's ability to move and efficiently construct the pipeline. In an urban area, conditions may require narrow working room but considerations must be given to where the contractor will string out the pipe before it's installed, how an excavator can move within the working area, and where dump trucks can come in and out to bring in or remove material.

Depth. Depth of cut should be minimized to decrease cost and difficulty of construction. Any increase in depth may require larger working rooms and creates more spoil material and import material needed.

Existing Utilities. This has been mentioned previously in this paper but the value of locating existing utilities ahead of construction can not be overstated. Any utilities that are unknown to the contractor ahead of construction are likely to slow down construction, increase costs, and create safety risks.

Overhead Obstructions. Often overlooked, overhead obstructions can impact the speed of construction and create new safety concerns for the contractor. Overhead power and signalized intersections are the most common source of overhead obstruction and need to be observed carefully during construction and all OSHA requirements for working near these facilities must be observed.

CASE STUDIES

Allen-Plano-Frisco-McKinney (APFM) Pipeline. A thorough route selection process was used in the planning of the APFM Pipeline for the North Texas Municipal Water District. The four-phase project was 18.6 miles long, with the most critical segments completed in priority order. The first three phases included 13.2 miles of 72-inch pipeline. The design team reviewed multiple corridors in pre-design, some following a longer path to less developed road ways and others following a shorter route but in a fully developed corridor. The cost analysis showed that even though the cost per linear foot would be higher for the more congested route, ultimately the shorter pipeline would be less expensive. In the end, the pipeline alignment followed a larger parkway that enabled lane closures and use of a median. The project required heavy coordination with various municipalities.

Lessons learned from the project include:

- Cost Analysis – Determining the cheapest route is not always clear from the first look, it is worth taking the time to compare. For this project the more congested corridor became the least expensive but this is not always the case, spend the time to do a thorough cost analysis early on in the route selection.
- Avoiding Developing Areas – In some cases, designing around known conflicts can be easier than designing for unknown future conflicts.
- Project Phasing – phased construction can reduce impacts to the community and spread out the capital costs of the project.

Figure 1 – Allen-Plano-Frisco-McKinney Pipeline Construction down a median with multiple utility conflicts and partial road closures.

Regional Carrizo Program 36-inch Water Delivery Pipeline

The 11.5 mile, 36-inch pipeline crossed four counties and three cities to deliver water to the San Antonio Water System (SAWS) NACO pump station. The project was a fast tracked design and construction with many road and interstate crossings, railroad crossings, construction within a drainage channel, and other existing utilities that required significant coordination and planning.

Lessons learned from the project include:

- Utility Conflicts – several unforeseen utilities and utilities located in a different place than the record drawings indicated were encountered. More significant SUE work and construction contingencies would allow for a smoother construction process.

- Aerial Topographic Survey – Surface features were encountered during construction that were not visible from the aerial photography shot before the project. Even if a full topographic survey is not possible, limited ground survey in critical locations can help to reduce construction conflicts.
- New Development – New projects that were not identified during design created conflicts with construction. It's important to identify as many future projects as possible during design and leave a contingency for dealing with unforeseen new development when it is encountered during construction.

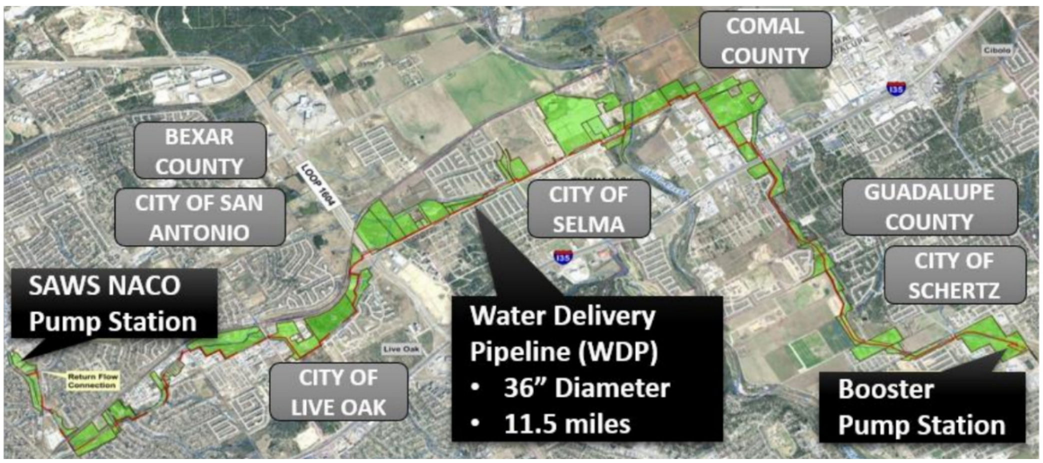

Figure 2 – Water Delivery Pipeline Alignment

CONCLUSIONS

The planning, design, and construction of a large diameter pipeline in an urban area comes with many challenges, but those challenges can be addressed if proper thought and foresight is given to the project ahead of time. Following the framework of the triple bottom line allows the engineer to address the economic, environmental, and social issues associated with pipeline construction. Additionally, spending more time on the front end of a project to identify all the risks and stakeholders involved will help to mitigate issues that can arise down the road. With proper planning and design, construction in urban areas does not have to be problematic, but instead can create lasting infrastructure solutions that are simple to maintain while serving the community for many years to come.

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Arching Effects in Box Jacking Projects

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Abstract

Box culverts are an essential component of highways and railroads since they transfer storm runoffs from upstream to downstream. Box jacking (BJ) is a trenchless technology method to install box culverts under embankments of existing highways and railroads with minimum surface disruptions. Over excavation (overcut) during box jacking operations is required to facilitate steering and reduce friction forces. Surrounding soils may collapse into the annular space during or after project box jacking and cause soil movements. Soil movement is reduced away from top of the box due to arching effect. Arching is a mechanism where soil particles are prevented from collapsing completely above the installed pipe/box and consequently less load is applied to pipe/box. The objective of this paper is to investigate the applicability of available empirical methods to estimate soil vertical displacements in box jacking projects and compare their results with a case study. Finite element modeling (FEM) using PLAXIS 2D is used to simulate box jacking operations. Data is collected from a box jacking project to validate the FEM model.

GROUND MOVEMENT ANALYSIS

Empirical methods are commonly used to evaluate ground movements. Empirical methods are based on mathematical relationships between measured values from previous projects. Statistical Regression analysis is widely used in empirical methods to find the relationships between project specifications such as pipe diameter, depth of pipe and soil properties and estimate soil deformation. O'Reilly and New (1982) conducted statistical analysis (regression analysis) on collected data to investigate vertical soil displacement on top of circular sections (e.g., pipe). They suggested an equation to determine maximum soil displacement using a parameter called trough (channel) width parameter; i. It was observed that maximum settlement (Smax) occurs exactly at the top of the opening and settlement magnitude decreases away from opening centerline. Moreover, inflection point distance forms opening (e.g., pipe/tunnel) centerline (i) increases as moving toward the ground surface.

Considering results from regression analysis, they suggested the following equation to estimate trough (channel) parameter, i.

$i_z = K.Z$

where:

i_z = Trough (channel) width parameter at depth z above tunnel axis (m)

K = A parameter that depends on the soil (e.g., i = 0.4 for strong clay and sand below water level, i = 0.7 for soft clay, and i = 0.2-0.3 for sand above water table).

Z = Depth of the tunnel from ground surface (m)

Mair et al. (1993) performed research to evaluate subsurface movements due to tunneling in clayey soils, and they showed that the normal distribution function can be adapted to estimate subsurface settlements trough (channel) by modifying the trough (channel) width parameter. However, they suggested that parameter K does not have linear relationship with depth.

$$i = K.(z_0 - z)$$

where,

K = A function of depth ($K = \frac{0.175+0.325.(1-\frac{z}{z_0})}{1-\frac{z}{z_0}}$)

z_0 = Depth of tunnel axis from ground surface (m)

z = Depth of the specific horizon from ground surface (m)

O'Reilly and New and Mair et al. suggested the following equation to calculate maximum vertical soil displacement, S_{max}.

$$S_{max} = \frac{V_S}{2.5\, i}$$

where:

V_s = Volume of surface settlement (m^3/m) ($V_s = \frac{\pi(d_S^2 - d_R^2)}{4}$)

i = Horizontal distance of the inflection point of the settlement trough from the tunnel centerline (m)

d_s = Outside diameter of the jacking or shield machine

d_R = Outside diameter of the jacking pipe

Mamaqani and Najafi (2014) collected and analyzed displacement data from box jacking (BJ) projects. Statistical Regression analysis was adopted to develop an empirical equation to estimate maximum surface settlement (S_{max}) in sandy soils with small amount of cohesion.

$$S_{surf,max} \text{ (mm)} = -0.58 + 2.5\, w + 0.49\, h + 0.18\, s - 0.36\, H + 0.21\, \gamma - 0.37\, c$$

Where:

w = Box width (m)

h = Box height (m)

s = Overcut size (mm)

H = Depth of box from ground surface (m)

γ = Soil Density (KN/m^3)

c = Soil Cohesion (KPa)

METHODOLOGY

PLAXIS 2D, geotechnical finite element modeling software (PLAXIS, 2011), was used to simulate BJ operation. To simulate a real BJ project procedure, stage construction feature was adopted. First stage of model analysis was generating initial stress due to soil weight and second stage of analysis, which was plastic analysis, was generated by activating box culvert and annular space, and deactivating soil inside the annular space and soils inside the box culvert. Deactivating annular space soil allows the soil to collapse into the annular space. Once the soil contacts the box culvert, the box stops further movement. To calculate the displacements associated with BJ operation, displacement due to initial stress generation reset to zero. Therefore, only displacements due to soil collapse into the annular space was captured. Figure 1 illustrates boundary conditions and parameters in PLAXIS models.

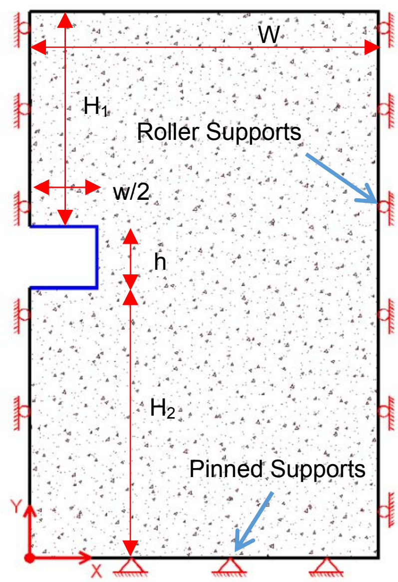


Figure 1. Boundary Conditions (Mamaqani, 2014)

Approximately 300 unique BJ project specifications were generated and modeled in PLAXIS to investigate vertical soil displacement above box culvert. Table 1 shows the minimum and maximum range of soil properties considered herein. Soil properties listed in the table are derived using Standard Penetration Test (SPT) relationships with modulus of elasticity (E), friction angle (ϕ), and unit weight (γ). Since there is no relation between SPT value and cohesion, a range of cohesion from 0 to 24 kPa (0 to 3.5 psi) is considered for soils in this study.

Table 1. Minimum and Maximum Soil Properties Considered in the Research (Mamaqani, 2014)

Property	Modulus of Elasticity, MPa (psi)	Friction Angle (Degree)	Cohesion, kPa (psi)	Unit Weight, kN/m^3 (lb/ft^3)
Min	9 (1,305)	30	0 (0)	14 (89.1)
Max	32 (4,640)	40	24 (3.5)	20 (127.3)

In this research, six groups of box culverts, as presented in Table 2, are considered. Box sizes are selected based on standard dimensions provided by manufactures catalogs.

Table 2. Considered Box Dimensions (Mamaqani, 2014)

No.	Width (Span), m (ft)	Height (Rise), m (ft)
1	1.8 (6)	1.2 (4)
2	1.8 (6)	1.8 (6)
3	2.4 (8)	1.2 (4)
4	2.4 (8)	2.4 (8)
5	3 (10)	1.5 (5)
6	3 (10)	3 (10)

To analyze the effects of box depth from ground surface to top of the box culvert on a surface settlement, different box depths ranging from 2h, 3h, 4h, 5h and 6h, where h is the height of the box, were considered. Installing box culverts at a depth of less than h is either not economical compared with open-cut method or requires special caution since it may causes large surface settlement. Since the depths of box culverts from surface are less than five times that of the yielding strip width (B_1), arching effect extends to the ground surface and is, therefore, considered in this study.

Since an overcut size of 25 to 50 mm (1 to 2 in.) is required to install box culverts, the overcut sizes of 30 mm (1.18 in.), 40 mm (1.57 in.), and 50 mm (1.97 in.) were used in this research.

CASE STUDY

The case study for this research was in the City of Vernon, northwest of Wichita Falls, Texas, under US Highway 287 (Figure 2). The purpose of this project was to alleviate the flood problem on the upstream side of the highway facility. TxDOT's Wichita Falls District decided to install a 1.8 m × 1.2 m (6 ft x 4 ft) box culvert to improve channel capacity at the depth of 6.7 m (22 ft) from the surface to top of the box. Geotechnical investigations including sieve analysis, standard penetration tests (SPT), and unconfined compressive strength (UCS) were conducted to determine soil properties.

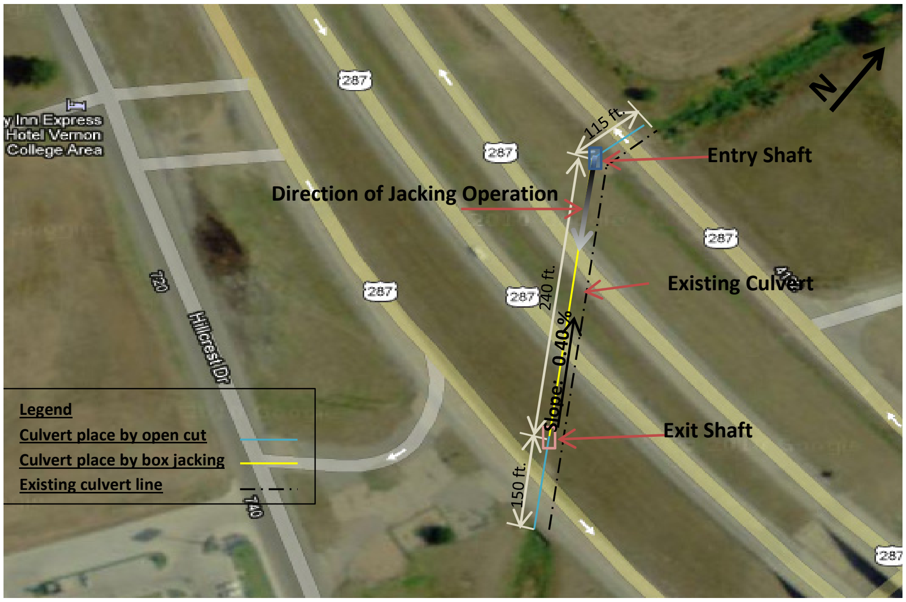


Figure 2. Vernon Project Location (Mamaqani, 2014)

Considering geotechnical reports and soil tests (e.g., sieve analysis, SPT, and Unconfined Compressive Strength (UCS)) results, soil properties was calculated using SPT relationships as presented in Table 3.

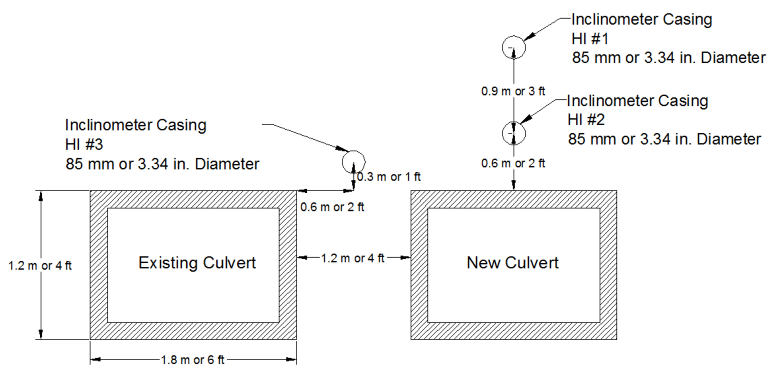
Table 3. Soil Properties of Vernon Project (Mamaqani, 2014)

ID	Depth m (ft)	Soil Type	N_{60}	Friction Angle (Degree)	Modulus of Elasticity, MPa (psi)	Unit Weight, kN/m^3 (lb/ft^3)	Cohesion, kPa (psi)
B1 (North)	0-1.2 (0-4)	SM	50	38	16.8 (2,436)	20 (127)	23 (3.3)
	1.2-4.8 (4-16)	ML	35	30	80 (11,600)	19 (121)	64 (9.3)
	4.8-12.2 (16-40)	SP	24	34	19.5 (2,827)	17.5 (111)	2 (0.3)
B2 (South)	0-12.2 (0-40)	SM	40	37	13.8 (2,001)	19 (121)	15 (2.2)

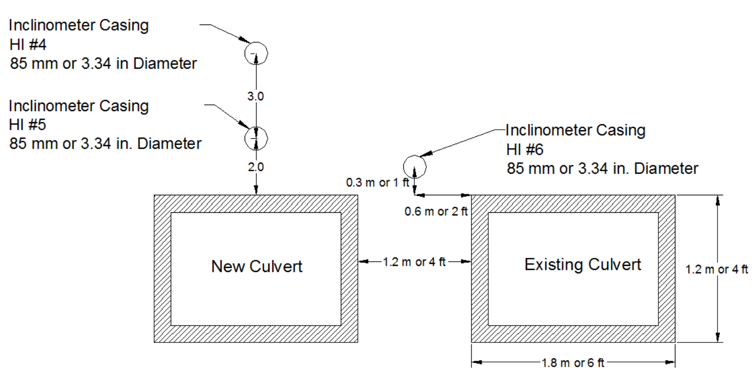
A Total Station TC407 survey instrument was used to measure the existing pavement surface to record settlement and/or heave at specific shoulder points. Also, a Horizontal Inclinometer (HI) system by Durham Geo Enterprises, Inc. was used to monitor settlement and/or heave around existing and new culverts (Figure 3).

Figure 3. Horizontal Inclinometer Data Collection System (DGSI, 2013)

To measure the soil movement in the vicinity of the box jacking operation, three 85 mm (3.34 in.) casings were installed on each side of the highway for inclinometer testing. The location of each casing in both the North and South sides are presented in Figure 4 (a) and (b) respectively.



(a)



(b)

Figure 4. Casing Locations; a) North Side, and b) South Side (Mamaqani, 2014)

RESULTS

Stress measurement on top of box culvert indicted that vertical stresses changed after box installation. This is because the surrounding soils collapsed into the annular space (overcut). Collapsing of soil into the annular space creates an active arching which causes the load, due to soil prism weight above the culvert, to reduce (Terzaghi, 1943) as illustrated in Figure 5. Arching effect causes less ground loss to be transferred to the surface and consequently less displacement occurred than on top of the box culvert at the ground surface.

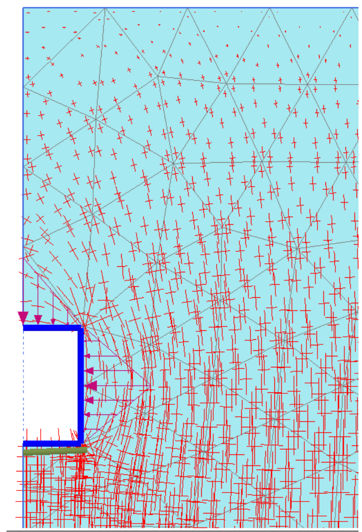


Figure 5. Stress Redistributions above Box Culvert (Mamaqani, 2014)

Considering soil vertical displacement distribution in depth, it was observed that the distribution has reverse relationship with depth of desired point. The following equation is suggested by the author to estimate vertical displacement at different depths above box culvert.

$$S_{max} = \frac{\tan\,(45 - \frac{\phi}{2})}{(H - H_0)} + S_{surf,max}$$

Where:

S_{max} = Vertical Displacement at depth H_0

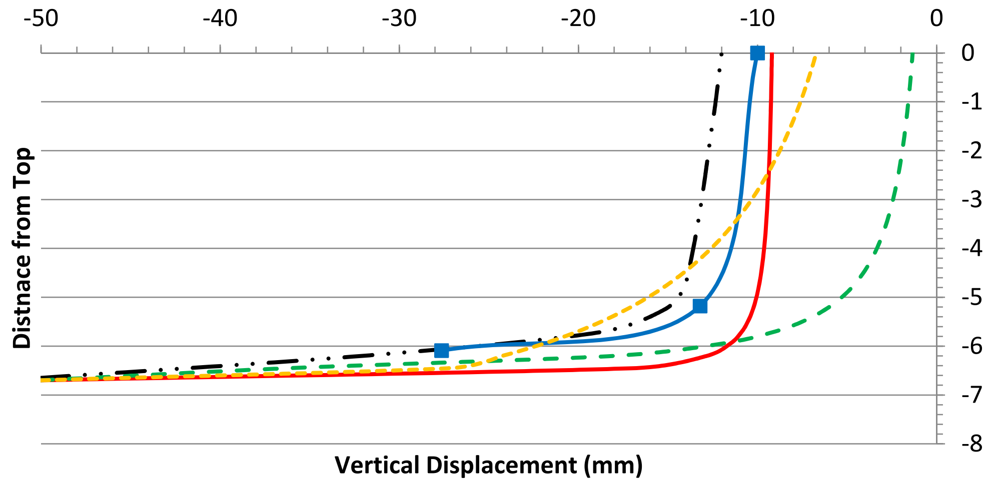
H = Depth of box from ground surface

H_0 = Depth of desired point to calculate vertical displacement

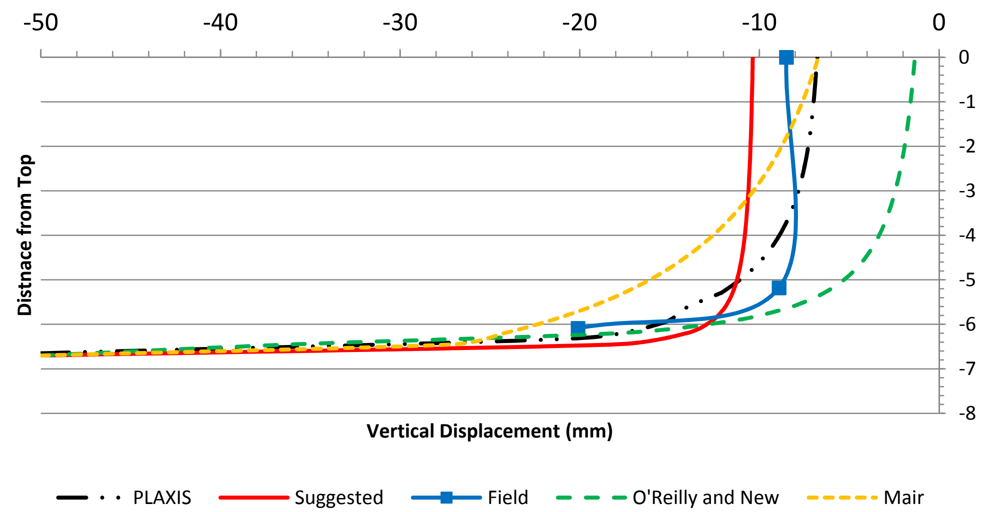
$S_{surf,max}$ = Maximum surface displacement

The main differences between suggested equation and the one suggested by O'Reilly, New and Mair is K. In the suggested equation K is a function of soil friction angle and is defined as 1 / tan (45+ϕ/2) while in O'Reilly and New K is a constant number and in Mair equation depends on depth.

Two scenarios (Scenario 1, and Scenario 2) were generated in PLAXIS 2D to investigate soil displacement. Figures 9 (a) and (b) compare vertical soil displacement distribution over the depth obtained from PLAXIS model, suggested equation and field measurements.



(a)



(b)

Figure 6. Vertical Displacements Comparison between PLAXIS, Suggested Equation, Field Data, O'Reilly and New, and Mair; a) Scenario 1, and b) Scenario 2

It was observed that vertical displacement decreases away from the top of the box culvert until it reaches its minimum value on the surface. Vertical displacement was diminished immediately in almost 1.8 m (6 ft) above the box culvert and then it continued to decrease gradually to the surface.

CONCLUSIONS

Results from this paper showed that arching happens above box culverts regardless of its rectangular shape and prevents soil from collapsing completely on top of box culverts. It was observed that soil displacement is reduced significantly at top of the box culvert in an area with the height equals to box culvert width and then gradually decreases away and reaches its minimum at the ground surface.

However, it was observed that the stress reduction above box culverts is less than pipe culverts. This is because stress can be transferred toward two sides on top of pipe culverts better than box culverts due to round section.

Acknowledgements

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Pipe Haunching Study Using Non-Linear Finite Element Analysis Including the Use of Soilcrete

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Abstract

Placing material in the hard to reach places underneath a pipeline called the haunches, is very difficult to do and very labor-intensive. This is one of the most important tasks when installing a pipeline. The main objective of pipe haunching is to provide firm uniform support to the pipe that will not change over time due to consolidation, moisture ingress, collapse etc. This is the main reason for specifying the use of a well-graded free-draining sand-gravel type of material in the pipe haunches. The purpose of this paper is to evaluate the effect of different haunching techniques (i.e., placement and compaction of material underneath the pipe in the haunches) on the predicted structural responses and performance of buried pipes using a non-linear finite element analysis program. Results are presented for the following typical installation conditions:

- No haunching (i.e., loose/dumped support such as 50% Standard Proctor density),
- Haunching (i.e., firmer support such as 85% Standard Proctor density),
- Placing silty sand (i.e., AASHTO A-2-4) material in the haunches,
- Placing well-graded sand-gravel material (i.e., AASHTO A-1-a, A-1-b) material in the haunches, and
- Using flowable low-strength soilcrete (i.e., 700 kPa compressive strength) in the haunches.

Results reported herein include predicted pipe structural responses for a DN1200 steel pipeline such as deflection, wall thrust, normal pressure and bending moment. Lastly, the importance of proper pipe haunching and the beneficial use of soilcrete are clearly demonstrated from the results of the parametric study.

INTRODUCTION

Placing material in the hard to reach places underneath a pipeline called the haunches, is very difficult to do and very labor-intensive. This is one of the most important tasks to accomplish when installing a new pipeline. The main objective of pipe haunching is to provide firm uniform support to the pipe that will not change over time due to consolidation, moisture ingress, collapse etc therefore the requirement for specifying a well-graded free-draining sand-gravel material.

PROBLEM STATEMENT

Utilize a non-linear finite element analysis (FEA) program capable of modelling the non-linear stress-dependent stress-strain behavior of real soils. Model different soil support conditions consisting of the following:

- No pipe haunching
  (i.e., loose/dumped support such as 50% Standard Proctor density),
- Pipe haunching
  (i.e., firmer support such as 85% Standard Proctor density),
- Use of silty sand bedding and backfill material
  (i.e., AASHTO A-2-4 or USCS GM, SM) and
- Use of flowable fill low-strength soilcrete
  (i.e., 700 kPa compressive strength maximum at 28 days).

DN1200 STEEL PIPELINE

A DN1200 steel pipeline with an 8 mm X42 wall thickness was selected for all the cases. The pipe diameter to wall thickness ratio (D/t) is 150 with a pipe bending stiffness of 274.4 kN/m/m based on $EI/(0.149R^3)$ and 5.2 kN/m/m based on EI/D^3. From a pipe handling and flexibility point of view the pipe is considered quite stiff with typical minimum pipe stiffness values of 2.0 kN/m/m based on the CIRIA recommendations (CIRIA, 1978). From experience, a minimum practical D/t of 160 or pipe stiffness of 4.0 kN/m/m is recommended for steel pipelines transported, installed and backfilled without installing any temporary props or spiders. When the latter is used, the maximum D/t can be greatly increased whilst pipe wall thickness is controlled by the maximum internal design pressure, external loading and pipe buckling capacity.

FEA SOFTWARE OVERVIEW

The use of FEA in our design environment is continuously increasing and FEA solution levels are getting more and more sophisticated and complex. It is therefore critical that FEA users are well trained and experienced whilst using well-proven FEA software solutions. One such FEA software is CANDE freely available in the public domain or incorporated into commercial packages such as CandeCAD Pro.

CandeCAD Pro

CandeCAD Pro (Culvert ANalysis and DEsign inside AutoCAD) is a specialized finite element software application developed specifically for the design, analysis and evaluation of buried pipes, culverts and other soil-structure interaction systems. CandeCAD Pro incorporates the widely used and accepted finite element source code of CANDE (Katona et al., 1976, and Musser, 1989) developed under sponsorship of the United States Federal Highway Administration (FHWA).

CANDE effectively models soil-structure interaction by various means such as incremental construction, interface slip, hyperbolic stress-strain relationships for soil, and simulation of compaction pressures. The program has been used successfully in the past to model regular and large-span culverts and buried pipe installations by various researchers and design consultants (Chang et al., 1980; McVay and Selig; 1982, Katona et al., 1979; Vaslestad, 1990; McGrath et al., 1999; Webb, 1999; Selig and McGrath, 1994 and Oswald and Furlong, 1993).

FEA Element Types

The pipe-soil structure is constructed of continuum quadrilateral and triangular soil elements, beam-column elements, and interface elements. The soil elements consist of either 3 or 4 external nodes with two translational degrees of freedom at each node (vertical and horizontal displacements) in addition to internal degrees of freedom.

Various soil models are available for representing stress-strain behaviour of the soil elements from linear elastic to nonlinear stress-dependent. The beam-column elements are bi-nodal with three degrees of freedom at each node, two translational and one rotational. Various models are available to represent different culvert and pipe material such as corrugated metal, reinforced concrete, plastic, and a basic model for nonstandard materials or built-up pipe materials. The models include nonlinear material behaviour such as metal yielding and concrete cracking and crushing.

Interface elements are used to model the interface between two subsystems such as the culvert wall and the surrounding soil, or the trench wall and trench fill. These elements allow for frictional sliding (interface shear force exceeds the product of normal force and friction coefficient), separation (interface normal force exceeds the tensile breaking limit), and re-bonding if additional loading brings the subsystem together again.

2D FEA MESH AND SOIL MATERIAL ZONES

Figure 1 below shows the geometry of the FEA mesh and the different soil material zones. In total, seven (7) different material zones were specified each consisting of unique material properties and stress-strain material behaviour (i.e., linear elastic, or non-linear stress-dependent). The soil material zones and soil material models are summarized in Table 1.

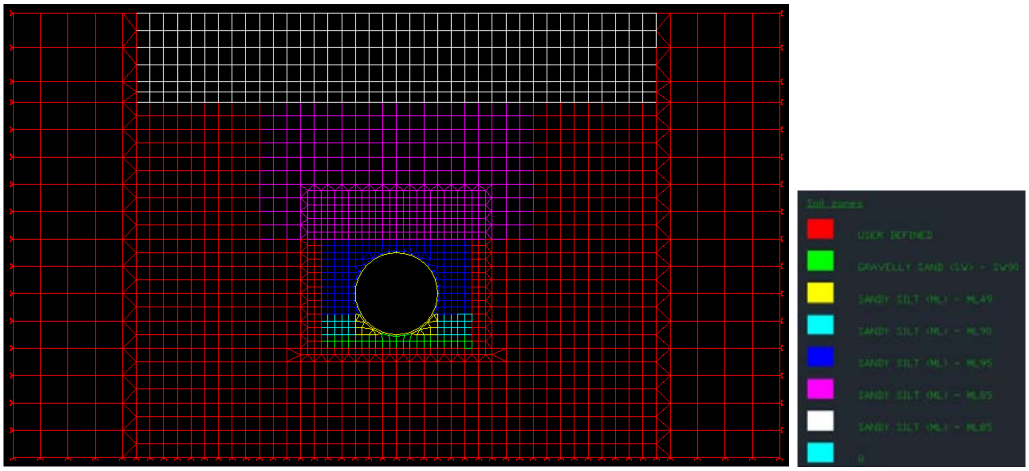

Figure 1. Non-Linear FEA Mesh and Soil Material Zones.

Table 1. Summary of FEA Soil Material Zones and Descriptions.

<table>
<tr><th>Soil Zone</th><th>Soil Description</th><th>Material Model</th><th>Description</th><th>Colour (Figure 1)</th></tr>
<tr><td>1</td><td>In Situ</td><td>Linear Elastic</td><td>Pre-existing material</td><td>Red</td></tr>
<tr><td>2</td><td>Bedding</td><td rowspan="6">Non-Linear Stress-Dependent</td><td>Imported bedding layers underneath pipeline</td><td>Green</td></tr>
<tr><td>3</td><td>Haunch</td><td>Imported material in pipe haunches / soil wedges underneath pipe springline</td><td>Yellow</td></tr>
<tr><td>4</td><td>120° Support Cradle</td><td>Material extending beyond haunches to trench wall but excluding Zone 3</td><td>Light Blue</td></tr>
<tr><td>5</td><td>Embedment (Cover 0.2 m)</td><td>Material surrounding pipe up to 200 mm cover but excluding Zones 3 and 4</td><td>Navy Blue</td></tr>
<tr><td>6</td><td>Backfill - Stage I (Cover: 2.2m)</td><td>Main trench backfill up to 2.2 m cover (arbitrarily selected)</td><td>Purple</td></tr>
<tr><td>7</td><td>Backfill - Stage II (Cover: 3.5m)</td><td>Main trench backfill above Zone 6 to final 3.5 m cover</td><td>White</td></tr>
</table>

The pipe was installed during Construction Increment No. 1 (CI1) together with the in situ material. As the natural in situ settlement has already taken place, the in situ material (Zone 1) was assigned zero unit weight (i.e., considers the net effect of filling the trench only). For the purpose of the parametric study, backfill material was placed sequentially in layers of 200 mm thickness alternating on the sides of the pipe to avoid creating unbalanced loading. Also, fifteen layers were used to place the trench material to avoid large load steps and possible material convergence issues as shown in Figure 2.

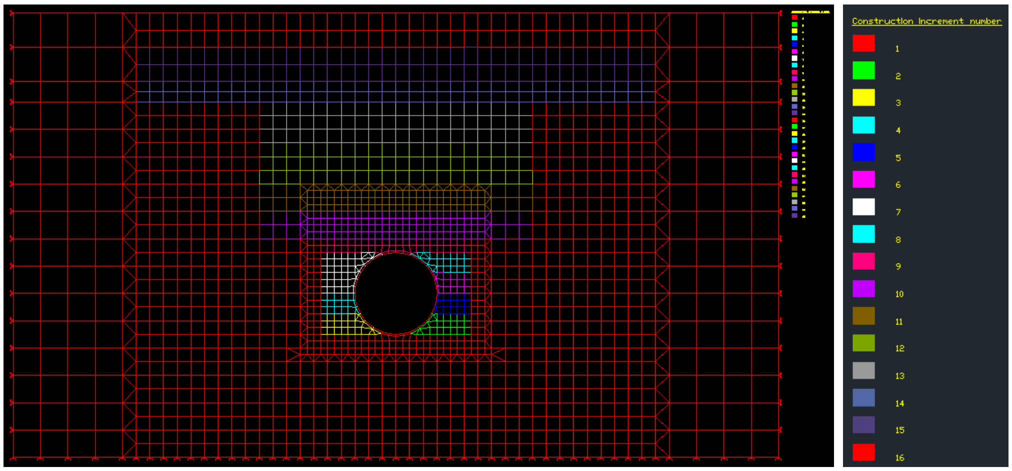

Figure 2. Incremental Construction Modeling and Backfill Placement Layers.

SIMULATION CASES

Table 2 below summarizes five parametric case studies simulated with the FEA model to investigate the haunching requirements.

Table 2. Summary of Parametric FEA Cases – Pipe Haunch Investigation.

		Backfill Material Zones						
Case	**Brief Description**	**Soil #1: In Situ**	**Soil #2: Bedding**	**Soil #3: Haunch**	**Soil #4: 120° Cradle**	**Soil #5: Embedment**	**Soil #6: Backfill: 2.2m Cover**	**Soil #7: Backfill: 3.5m Cover**
H1	Imported SW bedding; No haunching; Silty Sand backfill	Linear Elastic	SW90	ML50	ML90	ML90	ML85	ML85
H2	Similar to H1 above. Haunching;	Linear Elastic	SW90	ML90	ML90	ML90	ML85	ML85
H3	Similar to H1 above. Silty Sand bedding;	Linear Elastic	ML90	ML50	ML90	ML90	ML85	ML85
H4	Soilcrete Bedding, Haunches & 120° Cradle	Linear Elastic	Soilcrete	Soilcrete	Soilcrete	ML90	ML85	ML85
H5	Silty Sand Bedding, Haunches, Cradle & Embedment at 85% Std Proctor: No haunching (ML50) (Not saturated)	Linear Elastic	ML85	ML50	ML85	ML85	ML85	ML85

Case H1 forms the base case for the comparisons and consists of placing an imported well-graded sand-gravel (GW, SW) bedding layer of 200 mm thickness in the bottom of the trench and compacted to 90% Standard Proctor density (SPD) (equivalent to 85% MOD AASHTO maximum dry density). Poor haunching is simulated by specifying a loose (dumped) compaction density of 50% SPD using a fine-grained silty sand (SM, GM) material according to USCS or an AASHTO A-2-4 material. Silty sand (SM, GM) material compacted to 90% SPD is used for the 120 degree bedding support cradle as well as for the pipe embedment material (Zone 5). Similar material compacted to slightly lower density of 85% SPD (conservative) is specified for the main trench backfill (Zones 6 and 7).

RESULTS

Horizontal and Vertical Soil Stress

Predicted soil horizontal and vertical stresses are presented in Figures 3 and 4, respectively for illustration purposes and for the final construction increment (CI16) only (i.e., soil cover of 3.5 m).

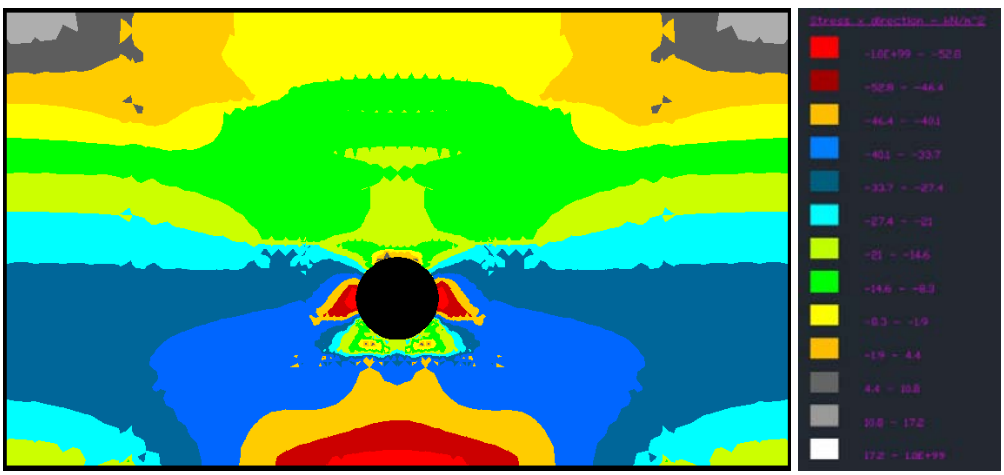

Figure 3. Case H1 - Predicted Horizontal Soil Stresses.

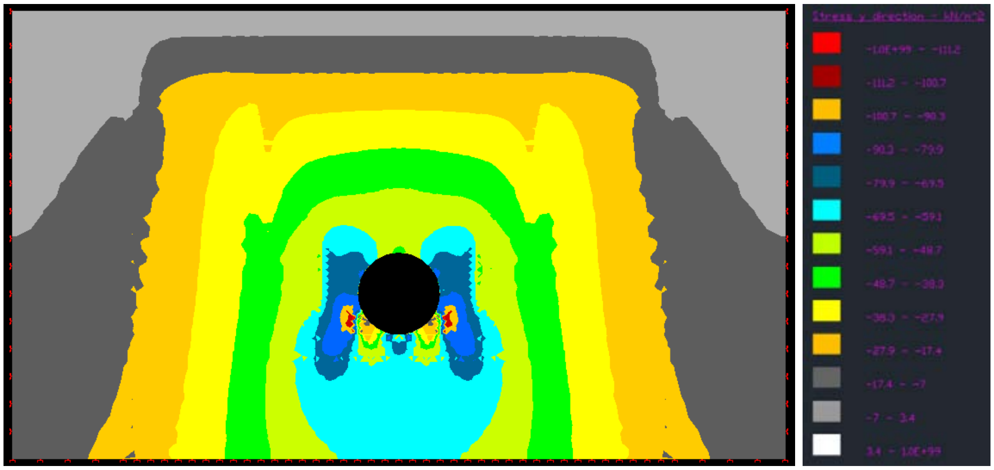

Figure 4. Case H1 - Predicted Vertical Soil Stresses.

Note that the magnitude of stress is a function of the colour intensity with maximum stresses of about 60 kPa and 120 kPa for Figures 3 and 4, respectively. Figures 3 and 4 clearly depict the soil stress distributions mobilized to provide the required support to the pipe and soil overburden.

Horizontal and Vertical Soil Strain

Similarly, Figures 5 and 6 present the predicted soil horizontal and vertical strains for the final construction increment (3.5 m of cover). In Figure 5 below, the maximum compressive strain represented in bright red equates to >0.26% whilst the dark red zones represent strain levels of between 0.08% and 0.26%.

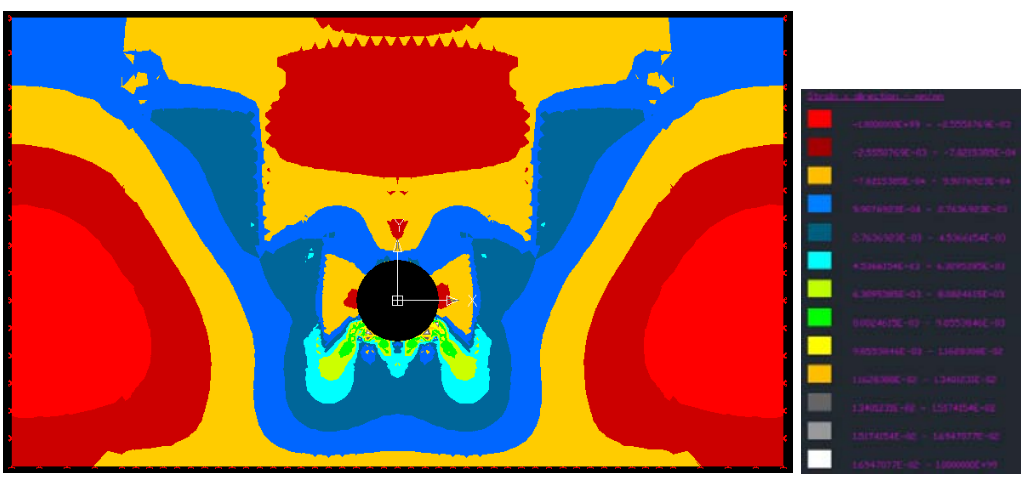

Figure 5. Case H1 - Predicted Horizontal Soil Strains.

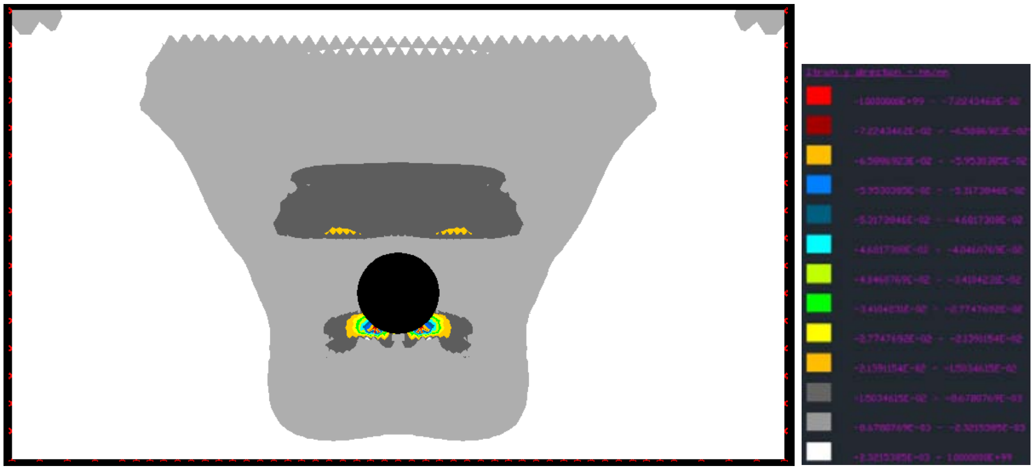

Figure 6. Case H1 - Predicted Vertical Soil Strains.

It is interesting to note the soil strain bulbs underneath the pipe in the region of the pipe haunches illustrated in Figure 6 above. As expected, the largest vertical soil compressive strain (i.e., varying from 1.5 % to 7.2%) occurs underneath the pipe in the haunches due to the very compressible nature of the haunch material essentially providing very little support. The white and light grey zones represent vertical strain levels as low as 0.2% to 0.9%.

Pipe Normal Pressure Distribution – Case H1

The predicted normal pressure distribution around the pipe is presented in Figure 7. As expected, the normal pressure distribution is not uniform varying quite noticeably around the pipe circumference with increased contact pressures at the pipe invert due to the compressible haunch material. The predicted pressures in the haunch zone decrease to as little as 15 kPa while increasing to 77 kPa near the invert. This variation in pressure will affect the pipe wall thrust too as will be shown next.

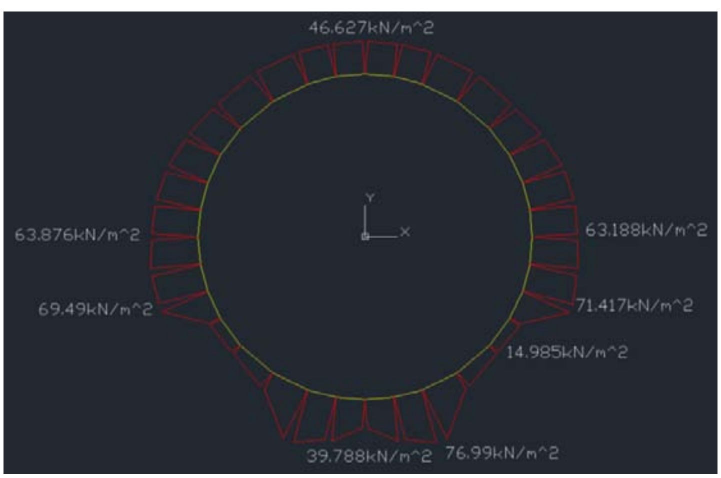

Figure 7. Case H1 - Predicted Pipe Normal Pressure Distribution.

Pipe Wall Thrust – Case H1

The predicted wall thrust around the pipe circumference during CI16 (soil cover of 3.5 m) is presented in Figure 8.

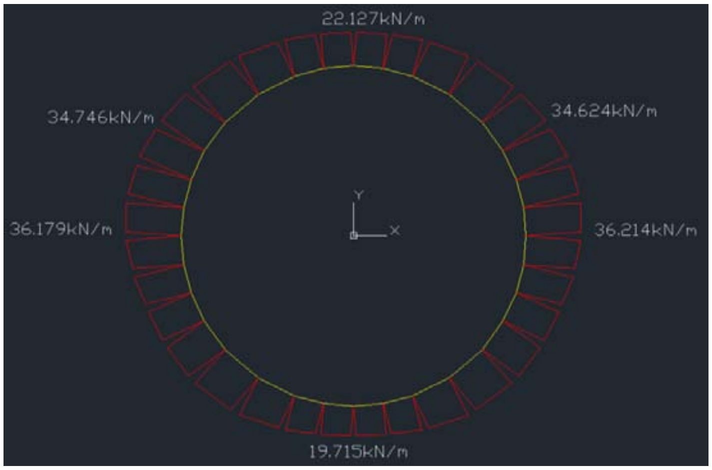

Figure 8. Case H1 - Predicted Pipe Wall Thrust.

The predicted pipe wall thrust varies around the pipe circumference with a maximum value of 36 kN/m at the pipe springline and reducing to 20 kN/m at the invert.

Pipe Wall Moment – Case H1

The predicted pipe bending moment distribution around the pipe circumference during CI16 (soil cover of 3.5 m) is presented in Figure 9.

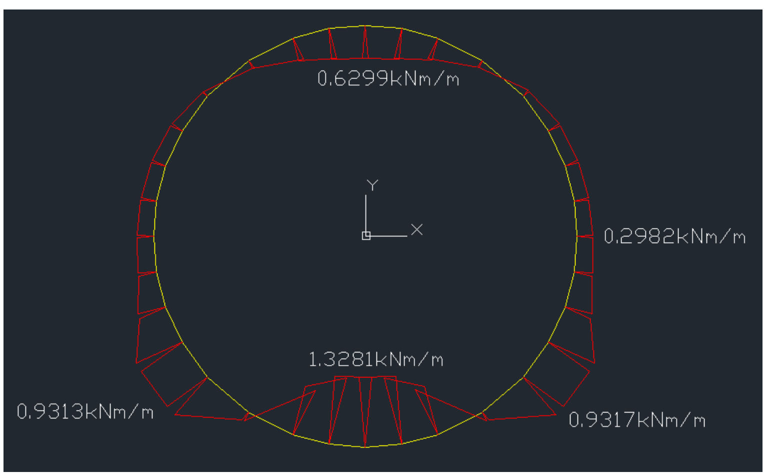

Figure 9. Case H1 - Predicted Pipe Wall Bending Moments.

The maximum positive bending moment (tension on inside fiber) occurs at the pipe invert with a value of 1.33 kN/m/m followed by the crown with a value of 0.63 kN.m/m. Note that positive moment is tension on the inside fiber and moment is plotted on the tension side of the pipe in Figure 9. Similarly, the maximum negative moment occurs at the pipe haunch with a value of 0.93 kN.m/m.

Pipe Deflections and Soil Displacement – Case H1

Figure 10 presents the predicted pipe deflection and resulting soil displacement during CI16 (soil cover of 3.5 m).

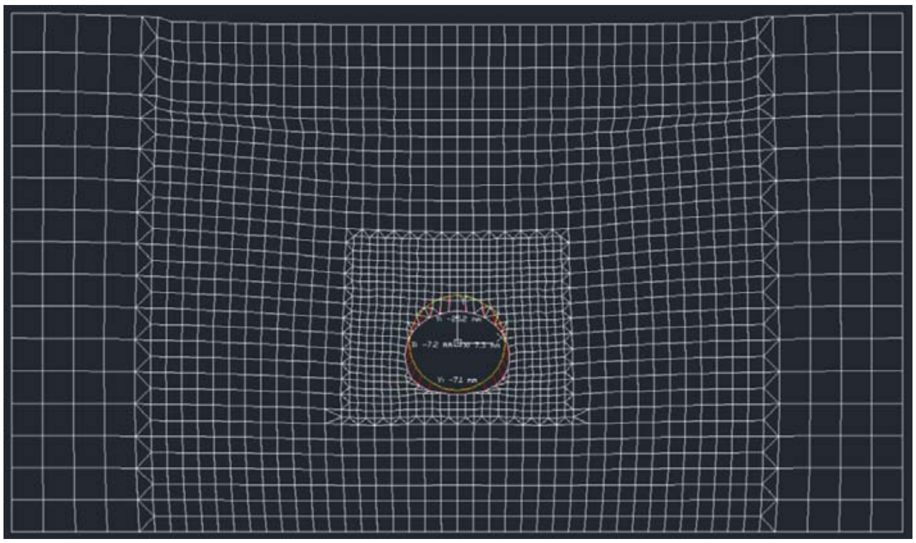

Figure 10. Case H1 - Predicted Pipe Deflection.

The pipe is predicted to settle 7.1 mm due to compression and settlement of the underlying soil layers. The net pipe vertical deflection is therefore 18.1 mm or about 1.48 % under 3.5 m of soil cover. Similarly, the horizontal deflection increase is 14.5 mm or about 1.19%.

Pipe Wall Strain and Stress

Maximum and minimum predicted pipe wall strains and stresses during CI16 are summarized in Table 3 below.

Table 3 Predicted Pipe Wall Strain and Stress – Case H1 (No Haunching)

Inner-Fiber Strain	Outer-Fiber Strain	Strain Ratio Max-to-Yield	
Microstrain	Microstrain	%	
531	371	44%	**Max**
-404	-553	5%	**Min**

Factors of Safety

Predicted pipe factors of safety against wall thrust, displacement, elastic buckling and maximum bending stress are summarized in Table 4 below.

Table 4 Predicted Pipe Factors of Safety – Case H1 (No Haunching)

Predicted Factors of Safety					
Thrust	**Displacement at 5%**	**Displacement at 2%**	**Buckling**	**Bending Stress**	**Yield Strength**
61.41	3.3	1.3	31.14	2.32	43%

Case H2 – With Haunching

Selective results for Case H2 which includes pipe haunching are presented below in Figures 11 and 12 and Tables 5 and 6 at the maximum soil cover of 3.5 m. Haunching is modelled assuming that the material can be placed and compacted to 90% SPD.

Pipe Normal Pressure Distribution – Case H2

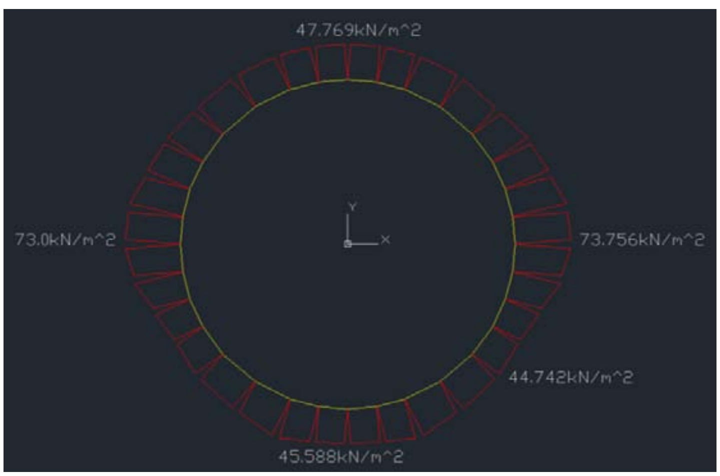

Figure 11. Predicted Pipe Normal Pressure Distribution for Case H2.

Unlike Figure 7 (no haunching), the normal pressure distribution is more uniform with increasing pressures at the pipe springline locations of 74 kPa. The predicted pressures in the haunch zone are higher too (45 kPa compared to 15 kPa before) while invert pressures reduce to 46 kPa compared to 77 kPa before without haunching.

Pipe Wall Moment – Case H2

The predicted pipe bending moment distribution around the pipe circumference is presented in Figure 12.

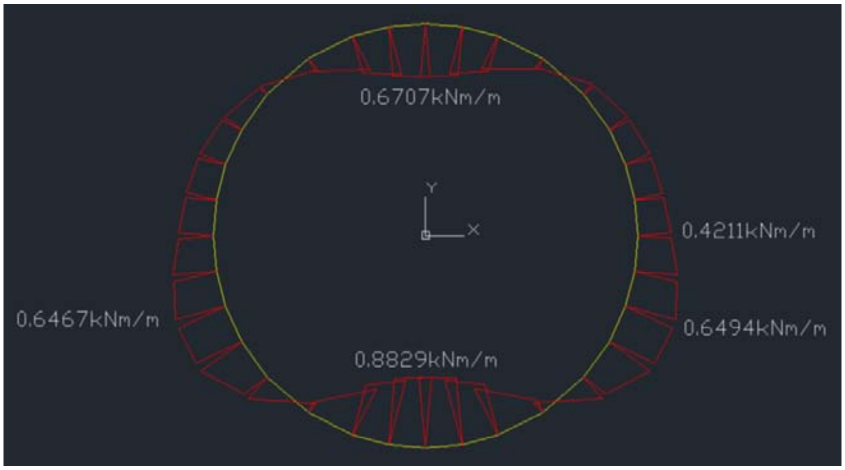

Figure 12. Predicted Pipe Wall Bending Moments for Case H2.

The maximum positive bending moment (tension on inside fiber) occurs at the pipe invert with a much reduced value of 0.883 kN/m/m compared to 1.3 kN/m/m before (33.5% reduction) followed by the crown with a value of 0.67 kN.m/m (essentially unchanged). Similarly, the maximum negative moment occurs at the pipe haunch with a much reduced value of 0.65 kN.m/m compared to 0.932 kN.m/m before (30% reduction).

Pipe Deflections and Soil Displacement – Case H2

The net predicted vertical pipe deflection is 16.2 mm or about 1.3 % under 3.5 m of soil cover compared to 18.1 mm (1.48%) before without haunching. The relevant change is small. The predicted horizontal deflection is exactly the same as before (14.5 mm).

Pipe Wall Strain and Stress – Case H2

Maximum and minimum predicted wall strains and stresses at 3.5 m of soil cover are shown in Table 5 below.

Table 5 Predicted Pipe Wall Strain and Stress – Case H2 (Haunching)

Inner-Fiber Strain	Outer-Fiber Strain	Strain Ratio Max-to-Yield	
Microstrain	**Microstrain**	**%**	
349	245	29%	**Max**
-285	-372	6%	**Min**

Factors of Safety

Predicted pipe factors of safety against wall thrust, displacement, elastic buckling and maximum bending stress are summarized in Table 6 below.

Table 6 Predicted Pipe Factors of Safety – Case H2 (Haunching)

Predicted Factors of Safety					
Thrust	**Displacement at 5%**	**Displacement at 2%**	**Buckling**	**Bending Stress**	**Yield Strength**
55.82	3.7	1.5	29.35	3.48	29%

Case H4 – Soilcrete Pipe Support

Selective results for Case H4 which allows for placement of soilcrete in the bedding, haunches and 120° support angle are presented below in Figures 13 and 14 and Tables 7 and 8 at the final soil cover of 3.5 m.

Pipe Normal Pressure Distribution – Case H4

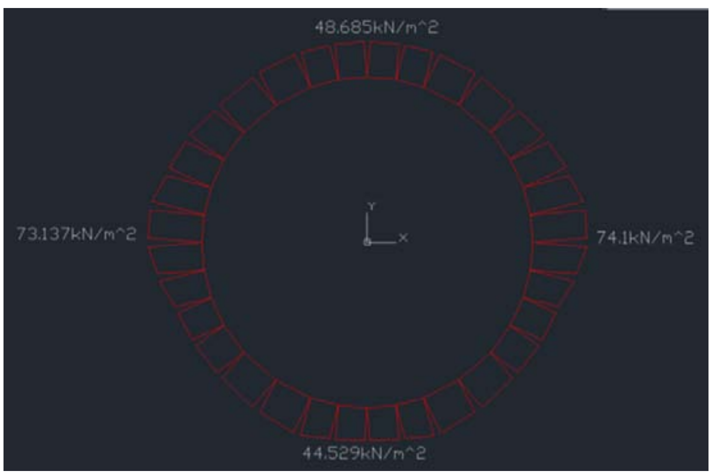

Figure 13. Predicted Pipe Normal Pressure Distribution for Case H4 (Soilcrete).

The predicted normal pressure distribution is very similar to Case H2 which included pipe haunching to 90% SPD.

Pipe Wall Moment – Case H4

The predicted pipe bending moment distribution around the pipe circumference is presented in Figure 14.

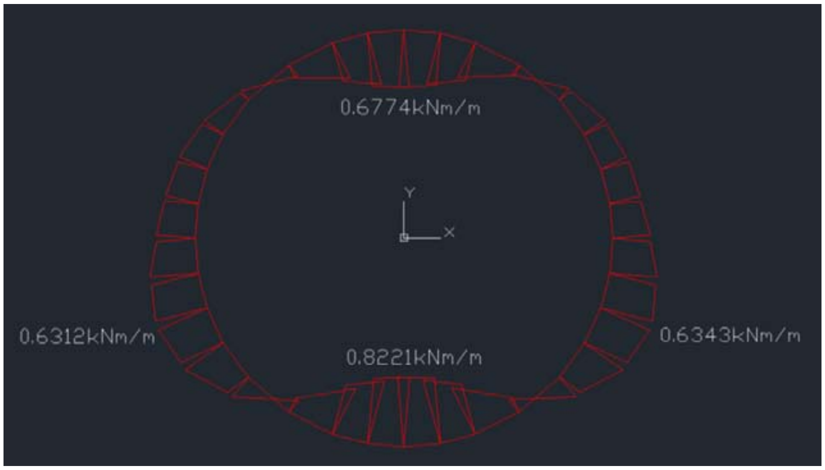

Figure 14. Predicted Pipe Wall Bending Moments for Case H4 (Soilcrete).

The predicted pipe bending moments are very similar to Case H2 which included pipe haunching to 90% SPD. Although not plotted, the predicted vertical and horizontal pipe deflections are similar to Case H2.

Pipe Wall Strain and Stress – Case H4

Maximum and minimum predicted pipe wall strains and stresses are summarized in Table 7 below. The values are slightly lower compared to Case H2.

Table 7 Predicted Pipe Wall Strain and Stress – Case H4 (Soilcrete)

Inner-Fiber Strain	Outer-Fiber Strain	Strain Ratio Max-to-Yield	
Microstrain	**Microstrain**	**%**	
324	238	27%	**Max**
-280	-347	6%	**Min**

Factors of Safety

Predicted pipe factors of safety against wall thrust, displacement, elastic buckling and maximum bending stress are summarized in Table 8 below.

Table 8 Predicted Pipe Factors of Safety – Case H4 (Soilcrete)

Predicted Factors of Safety					
Thrust	**Displacement at 5%**	**Displacement at 2%**	**Buckling**	**Bending Stress**	**Yield Strength**
54.72	3.8	1.5	29.16	3.74	27%

SUMMARY

1) The importance of pipe haunching was clearly demonstrated by this parametric study. Although the different haunching techniques were not specifically described herein, they may include shovel slicing and rod tamping.

2) Haunching is very effective in reducing the pipe invert moment by as much as 33.5%.

3) Use of soilcrete can reduce the pipe invert moment by an additional 4.5% compared to haunching alone.

4) Predicted pipe bending stress may be as high as 43% of the specified minimum yield strength (SMYS) of the material due to no haunching.

5) However, by specifying haunching and by replacing the material with soilcrete can reduce the maximum bending stress to 29% and 27% of SMYS, respectively.

6) Predicted pipe inner-fiber strains reach 531 and -404 microstrain, while outer-fiber strains reach 371 and -553 microstrain when no haunching is done. These are easily reduced to 349 and -285 microstrain and 245 and -372 microstrain for the inner and outer fiber strains, respectively by haunching. Using soilcrete will produce very similar results compared to haunching.

7) Both haunching and soilcrete will provide much more uniform pipe support as evidenced by the predicted normal pressure distributions.

RECOMMENDATIONS

1) It is critical to ensure proper uniform support to the bottom of the pipe that will not change over time due to time-dependent material behaviour (i.e., settlement and consolidation), possible migration of fines (i.e., loss of soil support) or moisture changes (i.e., collapse) all of which may affect the long-term structural stability and durability of the pipeline and its protective linings and coatings.

2) Haunching is critical and all efforts should be taken to ensure that a material meeting the above requirements is properly placed and compacted in the pipe haunches while working in thin layers and without damaging the pipe coating. A free-draining well-graded sand-gravel material meets these requirements. Also, such material requires less compaction energy to achieve the design soil stiffness and strength compared to finer grained and poorly

graded soils. The risk of damaging the external pipe coating and the amount of compactive effort required both increase with decreasing soil quality.

3) In lieu of the above requirement, it may be quite feasible and practical to utilize excavated trench material in many instances in a 3 to 5 % soilcrete mix for the pipe bedding, haunches and 120° support cradle. Controlled low strength material (CLSM) also known as soilcrete, flowable fill, controlled density fill, and flowable mortar, has been used as structural backfill for many years and for reasons that include:

   a) Ease of placement in hard to reach places (haunches) or in narrow trenches where space is limited,
   b) Fast backfilling operations since soilcrete is not compacted or tested for compaction requirements,
   c) Readily available from most ready-mix suppliers or mixed on site, and
   d) Ability to be removed if correct mix design is used.

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Trenchless Rehabilitation Saves Grottoes, VA, Culverts—and Money—Without Disrupting Traffic

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Abstract

Grottoes, Virginia discovered severe corrosion in portions of their stormwater system during a routine annual inspection in 2013. A large set of elliptical CMP culverts didn't pass inspection. The culverts were in poor condition with severe corrosion. Individual sections were failing and misaligned, and the town's consultants recommended replacement. Complicating the issue, the failing pipes were four parallel culverts which are all are quite large, 70" by 44", running directly underneath Dogwood Avenue, one of Grottoes' two main thoroughfares. The town obtained cost estimates for trench-and-replace from Brunk & Hylton Engineering, Inc. and, as expected, the price was high and the plan called for significant and lengthy traffic disruptions. Fortunately, Grottoes Town Manager Jeff Nicely had seen a trenchless rehabilitation process called CentriPipe that looked like it could be useful in this situation. In researching the solution, Nicely discovered the project cost was 15 percent less than the dig and replace estimate they had received, and had the added benefit of eliminating weeks of traffic disruptions. This paper will review the breadth of aging infrastructure situation that state and local agencies in the United States are facing, the engineering considerations in addressing failures, and the process, quality control measures, and results of the critical project in Grottoes, Virginia.

INTRODUCTION

A report published by the Midwest Regional University Transportation Center College of Engineering, Department of Civil and Environmental Engineering at the University of Wisconsin, Madison in 2008 provides insight into the number of culverts located underneath roadways and the impact on states and local agencies responsible for maintaining them. As the report states, the United States of America has the world's biggest transportation network system. The industrial growth during 1950s marked a rapid development in construction of high-speed, high-capacity roadway infrastructure. Today, the United States has 3,981,521 miles of roadway of which 46,726 miles belong to national highway system, 2,318,043 miles are paved roadway and 1,624,207 miles are unpaved roadway, which is the largest in the world. During the construction of these roadways, billions of culverts were installed under them. Since being installed, the location and condition of these pipes comes to notice generally only when there is a problem such as settlement or complete failure of a roadway.

The 2008 report further asserts that most of the states throughout the country are suffering from heavily deteriorating culverts, citing as an example, estimates by the Michigan Department of Transportation (MDOT) that there are about 200,000 culverts in the state of Michigan. As the 2008 report also notes, the Ohio Research Institute for Transportation and the Environment, at the University of Ohio made an important contribution in their report entitled "Risk Assessment and Update of Inspection Procedures for Culverts," (Mitchell et al, 2005). They introduced detailed culvert inspection system from data collected at sixty culvert sites. They reported that loss of culvert integrity could result in temporary roadway closure and considerable remediation costs and total collapse of culverts could result in a major safety risk for motorists. (Najafi, M., et al, 2008)

Clearly as these pipes reach the end of their useful life, state and local agencies must regularly inspect and repair or replace them. If not, these pipes are destined to fail and create a traffic danger. When culverts running underneath businesses and roadways are in need of repair, both failures and non-trenchless repair methods are disruptive to the local economy, causing serious hardship to business owners and individuals living and working in the community.

Sudden failures can cause a road section, parking lot, or building foundation to subside or collapse, thereby creating a sinkhole. Commercial areas have also been endangered when underground culverts fail – with responsibility often falling to local municipalities.

SITUATION OVERVIEW

Grottoes, Virginia is a town of 2,600 noted for its proximity to Grand Caverns, America's oldest show cave. In 2013, a different but significant underground asset – a large stormwater system – was found to have extensive corrosion.

The Virginia Department of Transportation (VDOT) gives the town funds for the maintenance of streets and stormwater networks, and as part of that arrangement, VDOT requires the town to conduct annual inspections. In the last report, a large set of elliptical CMP culverts didn't pass inspection. The consultant, Schwartz & Associates, told the town their culverts were in poor condition. The consultant informed Grottoes Town Manager, Jeff Nicely that "there was severe corrosion, and that individual sections were failing and misaligned." They recommended replacement.


Figure 1: Four parallel culverts under Dogwood Avenue

The prospect of repairing four large parallel culverts – 70” by 44” – directly underneath Dogwood Avenue, one of Grottoes’ two main thoroughfares, was a major concern. Shutting down Dogwood would cause very real problems for town residents and visitors. But repairs in this case were not optional, so finding a workable solution quickly became a top priority.

ENGINEERING AND DESIGN

Inspection is the first step in identifying critical issues. Understanding the size, shape, and material type of the original pipe, the local soil and water conditions, and the depth and load involved are all critical aspects when inspecting pipe condition, calculating strength requirements, evaluating rehabilitation methods, and designing for repair.

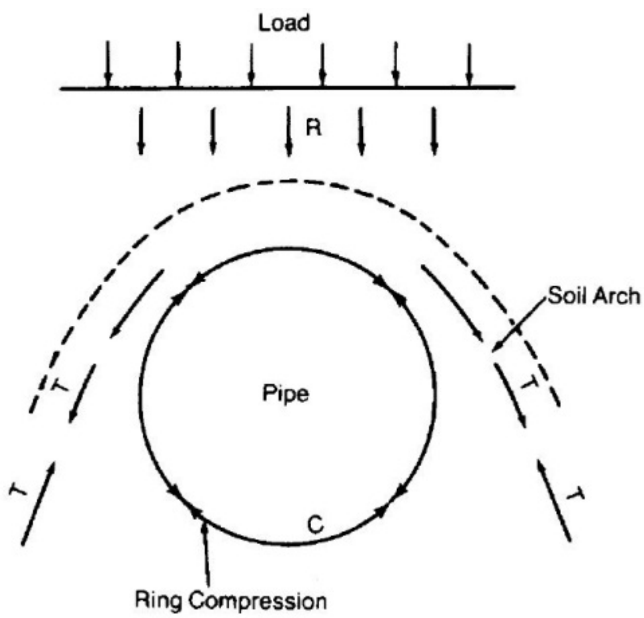

Figure 2: CMP Ring Compression Loading

In the case of flexible pipe materials (CMP, PVC, Welded Steel Cylinder, FRP, etc.), as the soil and surface loads are first applied the ring’s geometry tends to deflect somewhat in response to the magnitude of the loading and the resistance available from the embedment soil that has been placed around it. For a round pipe this deflection leads primarily to an elliptical shape with a decrease in the vertical diameter and an almost equal (slightly greater) increase in the horizontal diameter. This increase in horizontal diameter develops lateral soil support which, in turn, increases the load-carrying capacity of the pipe ring. The decrease in vertical diameter actually partially relieves the ring of some of the loading. The soil envelope around the pipe evolves quickly over the first few months of the new installation and takes on an arching reaction to the loads over the pipe (as illustrated in the Figure 2) – much like a masonry arch takes on its loading. Both the increase in the strength of the ring through the lateral soil resistance and the soil arching action contribute to the flexible pipe's in situ structural integrity. Because the modulus of the soil is far greater than the bending modulus of the flexible pipe, the resistance of the pipe to the applied dead and live loads present are borne almost exclusively by the surrounding soil.

The corrugated metal pipe arch (CMPA) is a commonly employed shape for culvert piping. This geometry has the ability to provide a single barrel opening that maximizes the hydraulic open area while minimizing the elevation of the hydraulic grade line (HGL). These single openings equate to easier maintenance for the owner as they are less likely to become clogged from any floating debris. The geometry of the ring is a composite of four arcs consisting of three different radii. Figure 3 depicts this shape and the initial load responses seen by the arch's cross-section. These reaction loads seen during the installation process shown at the corners and along the bottom are a function of the thrust loading coming onto the top radius and the ratio of the adjacent radii. Installation of pipes with this geometrical shape demands a very good foundation; and an essentially intact geometry over time is a testament to the quality of the foundation materials and their in situ performance to date.

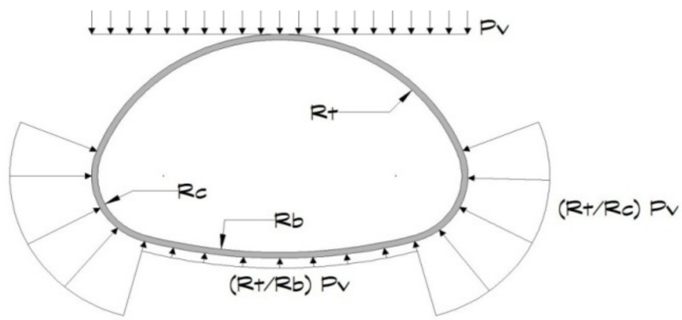

Figure 3: Initial Soil Loading Diagram for a CMPA

Corrosion is the typical deterioration mechanism for corrugated metal pipes. Corrosion usually initiates from the invert of the pipe spreading upward onto the sides of the pipe. The strength of the new lined pipe will depend upon its foundation consisting of the existing host pipe and embedment soil envelope, and thus the design of the liner is based upon the assessment of these two components and how they will likely respond to new loads coming on the pipe installation after lining. For installations where the invert is missing (a discontinuity in the pipe ring exists) but the geometry is essentially still intact, the engineer is presented with irrefutable evidence that the surrounding soil has taken on the portion of the thrust load that was previously carried by the CMP itself.

Where the geometry or shape of the existing pipe appears to be distorted from its original as-built shape, this could be an indication that the density of the pipe embedment has deteriorated (been compromised). In this case, the engineer is advised to undertake an investigation to determine the in situ density of the soil. If the density is at least 70% of the AASHTO T99 (standard proctor) density, the filling of any voids in the soil and restoration to the pipe ring can proceed on. If, however, the density of the embedment material falls below the 70% threshold, rehabilitation by lining of the pipe may not be feasible. The design engineer will in this instance need the input from a more thorough geotechnical investigation of the project site.

Following the inspection, depth of cover and soil type information is recorded and most often transmitted to a third-party engineer experienced in rehabilitation to recommend, design and sign off on a plan. When CCCP is the selected method – as was the case with the Grottoes project presented in this paper – relevant details including new pavement type and thickness, plus material and application notes are also included with the design.

PIPE AND ENVIRONMENTAL CONCERNS

As noted previously, the Grottoes project consisted of four large parallel culverts 112 feet long, for a total of 448 lineal feet, running directly under a main thoroughfare where traffic disruption would be a burden on the community. The culverts were found to be in poor condition with severe corrosion. Individual sections were failing and misaligned, however the CMP was not falling apart, so spot repairs and a new invert were not required prior to casting the new pipe. And fortunately the work was completed during a dry period so dewatering was not an issue.

Figure 4: Four large parallel culverts

CONTRACTING

The town obtained cost estimates for trench-and-replace from Brunk & Hylton Engineering, Inc. As expected, the price was high and the plan called for significant and lengthy traffic disruptions. Nicely then suggested a trenchless solution he had seen at a Rural Water Association conference called CentriPipe. After discussing it with the CentriPipe contractor in the area, Mike Shepherd, Nicely asked Brunk & Hylton to take a closer look at the trenchless system as a possible option for their specific situation.

The CentriPipe process is a centrifugally cast concrete pipe (CCCP) solution based on SpinCasting technology developed by AP/M Permaform. It was originally used in vertical applications, especially in manholes, but beginning in the 1990s the process has been refined for horizontal applications and is quickly becoming a standard for large diameter pipe and sewer rehabilitation. In essence, the CentriPipe® SpinCaster is pulled back through failing pipes while spraying very strong, highly adhesive, fiber-reinforced cementitious grout onto the pipe in thin layers. As the layers accrete, typically to a design thickness of around two inches, they form a new, structurally sound concrete pipe within the old pipe.

Figure 5: Minimal off-road staging area for equipment and materials

Figure 6: Trenchless repair leaves road open to traffic

The system has several advantages over competing solutions. Since it's an intrinsically structural solution, the structural strength of the failing sewer is immaterial—it just has to stay in place long enough to act as an outside form for the new concrete pipe to cure. And since the new pipe is thin, and adheres tightly to the existing pipe or culvert—the material used, PL-8000 from AP/M Permaform, adheres to metal, clay, brick, and HDPE—sewer flow capacity is minimally affected, and no annular space is left between the old and new pipes, so there is no ground or stormwater flow in that area. And CentriPipe is also cost-effective; prices are generally less than for other large-diameter rehabilitation methods.

After review by Brunk & Hylton, CentriPipe was selected for the Grottoes project for several signification reasons. First, compared just on a project cost basis, using CentriPipe was 15 percent cheaper than digging up the old sewers and replacing them. But that doesn't even account for the savings gained by not disrupting traffic for weeks, an even more attractive aspect for Nicely. He estimates that avoiding the costs of traffic monitoring saved another five percent or so, and that saving the town the hassles of disrupted traffic is an incalculable but significant benefit. Also, the lengthy permitting process may have been eased by the relative lack of disruption and excavation.

QUALITY CONTROL

Quality control for this project was performed in two ways. The thickness of the new pipe is the key factor, so the old CMP was measured from the top of the corrugations prior to rehabilitation, and again following rehabilitation to ensure the specified thickness had been achieved. Additionally, several holes were drilled along the new concrete, to verify thickness. These measures ensured that the work was completed to specification.

INSTALLATION

Figure 7: Spincaster and sled

The work was completed in two phases by Mike Shepherd's crew at D&S Contractors, and Arold Construction, both licensees of AP/M Permaform. The crews cleaned the culverts using the CentriPipe spincaster as a high-pressure washer to clear out debris that could affect adhesion. They then made multiple passes, pulling the spincaster on skids and pumping PL-8000 that was mixed on site (the material is dry, and delivered to the staging area in bags) to build up a final thickness of two inches. This dimension and other specifications were established by consulting engineers contracted by AP/M Permaform and based specifically on the conditions and requirements of the Grottoes project.

When the CentriPipe spincaster is pulled through the pipe, it evenly casts a centrifugally-compacted layer of PL-8000 into the interior of the pipe. The application head is retracted at the properly calculated speed to ensure an even thickness of PL 8000. A high strength, high build, abrasion resistant and corrosion resistant mortar, based on advanced cements and additives, PL 8000 has a compressive strength of 8,000 psi while standard concrete is typically 2,500 to 4,000 psi. When mixed with the appropriate amount of water, a paste-like material will develop which may be sprayed, cast or pumped into any area ¼ inch and larger. Two to four layers may be needed to achieve the appropriate design thickness which typically ranges from one to four inches. The hardened liner is dense and highly impermeable. The above stated performance is achieved by a complex proprietary formulation of mineral, organic and densifying agents and sophisticated chemical admixtures including rust inhibitors. Graded quartz sands are used to enhance particle packing and further improve the fluidity and hardened density. The composition also possesses excellent thin-section toughness, high modulus of elasticity and self-bonding. Fibers are added as an aid to casting, for increased cohesion and to enhance flexural strength.

CONCLUSION

Evaluating and repairing the aging storm and waste water systems managed by our municipal, county, state, and federal agencies is an on-going long-term prospect with very real challenges. And while infrastructure requires constant maintenance, agencies must also balance shrinking budgets and roads and other facilities that must stay open to serve the public. Selecting the best method for each situation requires experience and knowledge of the available solutions and the ability to inspect and assess pipe and environmental conditions to make the best possible rehabilitation decisions. Experience and knowledge are also critical to the successful installation of the selected methods.

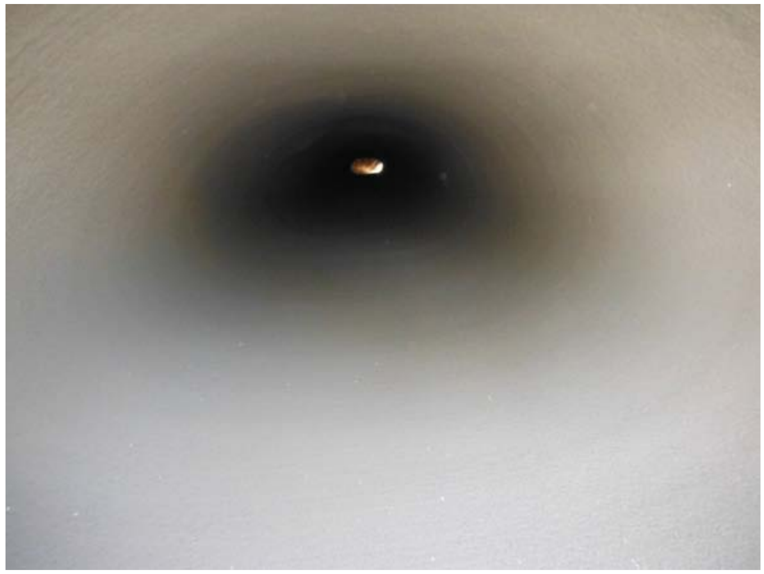

Figure 8: Smooth, seamless, watertight, and structurally sound rehabilitated culvert

In the case of Grottoes, VA, the town was able to renew existing culverts using the CentriPipe process to make them smooth, seamless, watertight, and structurally stronger than the old CMP, with longer projected service life. Nicely is quite happy with the results, and best of all, traffic never had to stop.

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Trenchless Technologies Decision Support System Using Integrated Hierarchical Artificial Neural Networks and Genetic Algorithms

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Abstract

The tunneling industry involves countless number of variables and complexities that have to be considered when selecting the construction method to be applied in different types of projects. In addition, the availability of different trenchless technologies makes it difficult to select the most suitable trenchless technology to be used. This paper introduces a framework for developing a Trenchless Technology Decision Support System (TTDSS) using a newly-introduced technique "Hierarchical Artificial Neural Networks (ANN)". The system integrates the concept of hierarchies with the ANN, taking into consideration the direct effect of the factors on each hierarchical selection. Sixty projects were introduced to the HANN, 80% of them were used as training cases and the remaining 20% were used for testing. Results indicated the potential of TTDSS in supporting trenchless technology specialists in their selection decisions, where the error percentage did not exceed 5%.

INTRODUCTION

Trenchless Technology (TT) is a collection of technologies and methods of subsurface construction for installation, rehabilitation or replacement of underground infrastructure systems with minimal surface disruption (McKim, 1997). In the past, the construction industry has been resistant to accepting new or unproven technologies and methods into projects. This can be attributed to the unknown risks associated with such new technologies and the lack of knowledge and understanding of its capabilities (Ueki, Haas, & Seo, 1999). Inappropriate utilization of TTs made stakeholders resistant to applying such technologies. As a result, there is a strong demand for intelligent models that are able to aid decision makers in their selection of TTs. With the rapid population growth and increasing subsurface infrastructure, TTs emerge to fulfill the need for rehabilitation and new construction. Developing a Trenchless Technologies Decision Support System

(TTDSS) taking into account user's project conditions will yield more reliable and rationale results. TTDSS should be useful to both new and experienced decision makers in the TT industry who are interested in choosing between a TT, optimized by the TTDSS for their conditions, or the traditional Open-Cut Construction (OCC).

TTs can be divided into Directional Trenchless Technologies (DTT) and Non-Directional Trenchless Technologies (NDTT). There are many types of TTs that fall under these two categories. This model will be concerned with 7 types of TTs that are subdivided into 4 major types of TTs, 2 major types under DTT and the other 2 under NDTT. Micro-Tunneling (MT) and Horizontal Directional Drilling (HDD) are types of DTT whereas Pipe Ramming (PR) and Horizontal Auger Boring (HAB) are types of NDTT. MT is a remotely controlled pipe jacking process where the MT machine is mounted with a guidance system for directional tunneling. MT uses fluid pressure to control excavation face stability and hydraulic jacks to push the machine forward. MTs are divided into Open face Micro Tunneling (OMT) and Closed face Micro Tunneling (CMT).

HDD consists of 2 stages in its application, drilling the directional pilot bore hole along the required path and pulling back the required pipe along the same path. The profile of HDD is usually an arc filled with slurry as the pilot bore is drilled. The slurry is then pushed into the soil as the pipe is pulled through. HDDs are divided into Mini Horizontal Directional Drilling Mini-HDD, Medium Horizontal Directional Drilling Mid-HDD and Maxi Horizontal Directional Drilling Maxi-HDD. HAB is a technique where two shafts are constructed, one for driving and the other for receiving the pipe. The auguring process involves excavating inside the steel casing that is continuously jacked. PR is like the HAB in that it has two shafts as well, one for driving and the other for receiving the pipe; however, it utilizes dynamic vibrations for installing the steel casing through the use of a hammer (Salem & Najafi, 2008).

Comparing TTs with OCC, OCC may be less expensive in the presence of favorable conditions and is applicable for all types of pipes; however, it requires more excavation and has a limited depth and applicability based on site conditions. As for TTs, they are favored over OCC, where minimal surface and subsurface disruption is important to the success of the installation. However, their main disadvantage would be cost if no restricting site conditions are present.

As shown in Table 1, TTDSS takes into account the following eight types of construction methods: OCC, OMT, CMT, Mini-HDD, Mid-HDD, Max-HDD, HAB and PR. Based on user-selected project conditions, the model decides the most convenient construction method for a project with the specified characteristics from a predetermined set of factors that influence selection. The model takes into account total project length, drive length, required accuracy, soil type, ground water impacts (e.g., de-watering), existing underground utilities, surrounding above-ground structures, work space requirements (e.g., street width), acceptable noise level, traffic impacts, pipe diameter, pipe material and pipe depth as the factors that most affect the selection of appropriate construction method.

Table 1. TTDSS Construction Methods

Abbreviation	Construction Method
OCC	Open Cut Construction
OMT	Open-Face Micro Tunneling
CMT	Closed-Face Micro Tunneling
Mini-HDD	Minimum Horizontal Directional Drilling
Mid-HDD	Medium Horizontal Directional Drilling
Maxi-HDD	Maximum Horizontal Directional Drilling
HAB	Horizontal Auger Boring
PR	Pipe Ramming

In this model, various factors were taken into consideration for system selection. For instance, the variations in drive length will definitely affect the choice of TT, or whether OCC will be sufficient for this project depending on a combination of the other factors. In addition, the required level of accuracy of the project is another vital factor that could impact the selection: e.g., where gravity lines may require a very high vertical accuracy, a Distribution network may only need medium accuracy, and the installation of cable lines may require the least accuracy. Moreover, soil condition includes the type of soil (sand, clay, silt, or rock and ground water existence). Furthermore, the presence of existing subsurface utilities will impact the choice of TT based on the attributes of the existing utility. It takes into account existing underground utilities such as network of gas pipes, high pressure gas pipes, crude oil, solar, normal voltage electricity cables, super high voltage electricity cables, or more than one type of utility.

Surrounding structures would also affect the TT used. Surrounding structures in the model included parks, historic areas, cemetery, residential development, industrial development, business development, or landscape area. Likewise, street width, subdivided into more than 4m and less than 4m, affects the choice as well. Besides, the noise and traffic levels are taken into consideration while choosing the most appropriate method. Pipe diameter is also an essential factor in TT as there are diameter limitations associated with the various techniques. In addition, pipe material is an essential factor that depends on the TT selected; where the model incorporates Reinforced Concrete Pipe (RCP), High Density Poly Ethylene (HDPE), Glass Fiber Reinforced Plastic (GFRP), Polymer Concrete Pipe (PCP), Vitrified Clay Pipe (VCP), Ductile Cast Iron Pipe (DCIP), Poly Vinyl Chloride pipe (PVC) and Steel Pipe (SP) as pipe material types. The pipe depth is a critical factor as well in determining the most appropriate TT.

LITERATURE REVIEW

Artificial Neural Networks (ANN) is an adaptive process, a mathematical model, a network of interconnected groups that process information, changing its structure during its learning phase. ANNs originated from central nervous systems, where they consist of nodes, the processing units, connected together similar to a biological network. Integrating ANNs with Genetic Algorithms (GA) alter the strength of network connections for optimized results. GA is a heuristic process similar to natural evolution. This process produces more reliable optimized solutions to search problems.

One of the models developed is a selection method (McKim, 1997) that utilizes a hierarchy based model. McKim's model divided the methods into specific components that define their capabilities and compares them to the required capabilities as per the characteristics of the project. This study did not incorporate the economic aspect in the method selection. Conducted in 1997, this study assumed the rather slow acceptance of stakeholders to new technologies like TTs. It included only TTs for repair and upgrade of existing infrastructure and facilities; however, it lacked TTs associated with installation of new infrastructure (McKim, 1997).

Other researchers developed a decision model for micro-tunneling method selection. This research tackled the increasing demand for appropriate TTs selection. Similar to McKim's research, this research was conducted early in 1999 when decision makers were not familiar with this technology and were not confident in its use due to the risks and costs that would be associated with the selection of an improper method. They developed a decision model that would select the micro-tunneling method of construction, then select the pipe type and choose the machine such that all selections were coherent. The method of micro-tunneling is based on the depth, diameter and drive length of pipe, ground-water table, site conditions, soil conditions, and existence and size of boulders. For pipe selection, the tool would select and list pipes in order of strength as per the user input data. Based on the input data and the pipe selected, the model would determine a recommended type of method that meets all of the user's requirements (Ueki, Haas, & Seo, 1999).

Another researcher developed Decision Support System software (Mathews & Allouche, 2012). The research tackled the growing concern regarding the constantly changing technologies through incorporating an up-to-date web-based source. This research resulted in the Trenchless Assessment Guide for Rehabilitation online web-based tool. The tool assesses the suitability of the new construction or rehabilitation method based on different types of pipelines, gravity sewers, sewer laterals, connection seals, pressure water pipes and manholes, where it uses the attributes included in the online database for the different categories according to defined parameters, length, depth, groundwater depth, diameter, grade and alignment accuracy, soil conditions and accessibility, to aid decision makers in their selection (Mathews & Allouche, 2012).

Several decision support systems for trenchless specialists were developed as highlighted above. However, the need for an extendable database of previous projects was highly recognized, especially for ANN-based models. Hierarchical modeling refers to the structural arrangement of parameters into multilevels where lower units belong to the hierarchy of successive higher units. The integration between the concept of hierarchies and ANN was the solution for guaranteeing an accurate decision-support system that would minimize the required running time, whenever extending the database. The system's objective is to act as a decision-support tool that will select the best construction method to be applied for each project based on pre-defined factors.

MODEL DEVELOPMENT

Model Framework

The TTDSS framework was inspired from the integrated relationship among several Hierarchical Artificial Neural Network (HANN) modules to reach the optimal decision. Figure 2 describes the general processes of the TTDSS and their interrelation. The framework features three different modules: (1) A Central Database Module that contains the projects data together with the implemented construction method for each project; (2) A HANN, which selects the result of each hierarchy based on pre-defined factors. This module is divided into 6 HANNs' that simultaneously work together to guarantee a high accuracy and precision for the system results; and (3) an optimization engine that minimizes the training error percentage between the ANN results and the actual outputs. As a result, the ANN module will set-out the weights among the input, hidden and output neurons to achieve a near optimum solution with a minimal error percentage.

Central Database Module

This database comprises a list of projects, along with their surrounding conditions and environment as shown in Table 2. Moreover, the central database includes the actual construction method applied for each project based on the expert provided database. The surrounding conditions and environment with the project requirements act as an input for the HANN in the following stage of the model process. Figure 1 shows a list of the factors taken into consideration in the TTDSS.

Table 2. Extract from the Central Database Showing Some of the Projects

Project/Factors	Inputs														
	Total Project Length (m)	Drive Length (m)	Usage (Accuracy)	Type of Soil	Ground Water Table Existence (De-	Existing Underground Utilities	Surrounding Above-ground Structures	Work Space Requirments (Street Width)	Acceptable Noise Level	Traffic Level (Impact)	Pipe Diameter (m)	Pipe Mateiral and Diameter Check	Pipe Material	Pipe Depth (m)	Pipe Depth and Pipe-line Usage Check	Project Type
Project 1	200	6	Gravity Lines	Gravel	Does not need De-watering	Network of gas pipes	Landscape area	More than 4 m	Low	High	0.6	Acceptable	Polymer Concrete Pipe (PCP)	3	Acceptable	Non-Crossing
Project 2	300	12	Carrier Lines	Clay	Needs De-watering	Solar	Park	More than 4 m	Medium	Medium	1.2	Acceptable	High Denisty Polyethylene Pipe (HDPE)	7	Acceptable	Non-Crossing
Project 3	600	24	Distribution Network	Cobles	Does not need De-watering	High pressure gas pipes	Historic Area (Antiquities)	Less than 4 m	Low	Low	3	Acceptable	[illegible]	[illegible]	Acceptable	Non-Crossing
Project 4	600	50	Cables	Rock	Needs De-watering	Super-High Voltage Electricity Cables	Historic Area (Antiquities)	Less than 4 m	Low	Low	1	Acceptable	High Denisty Polyethylene Pipe (HDPE)	3	Acceptable	Crossing

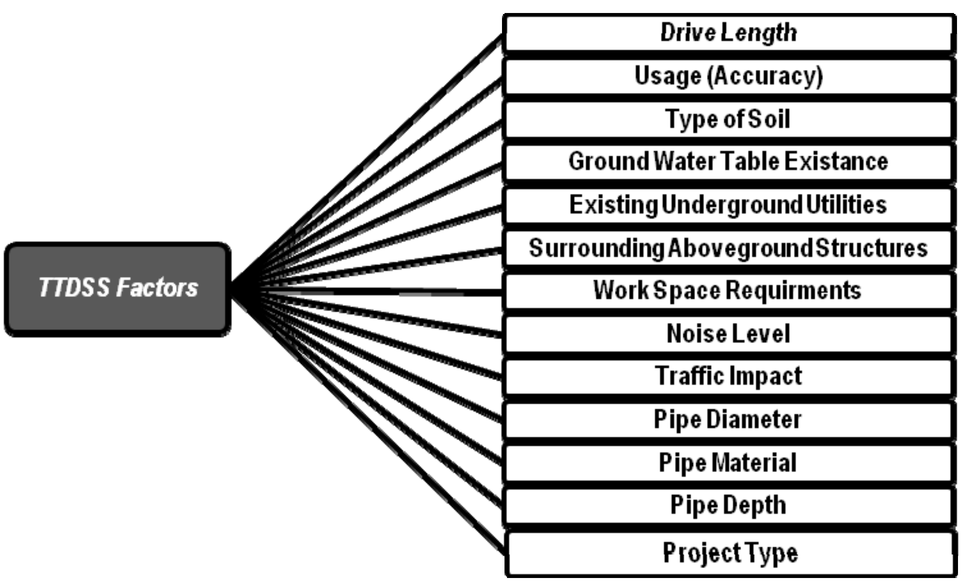

Figure 1. TTDSS key factors

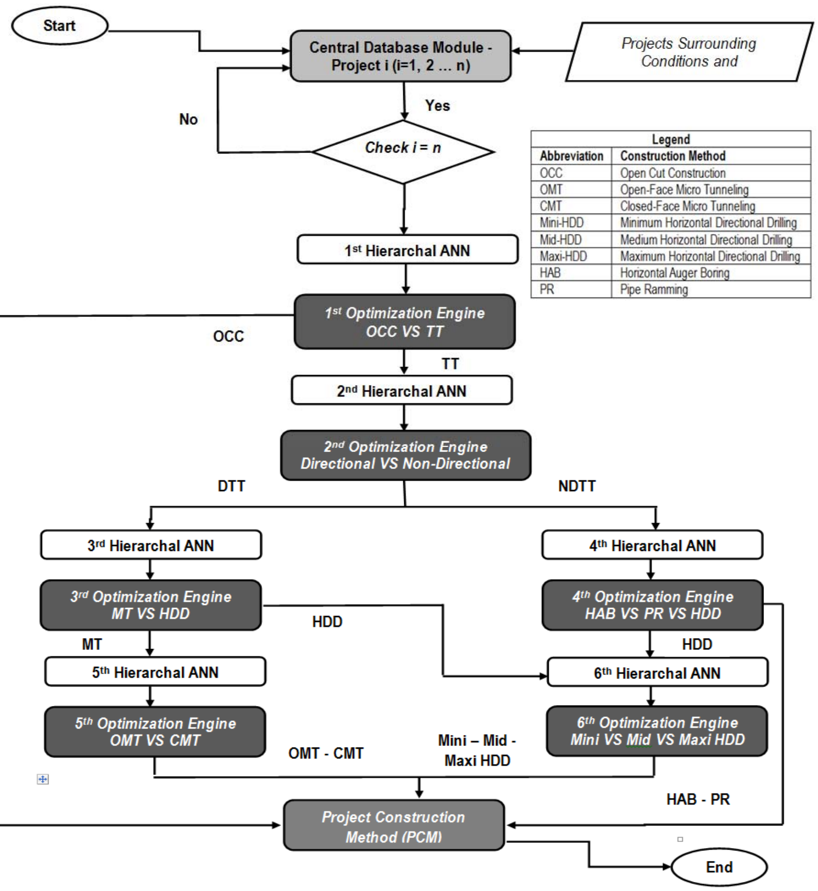

Figure 2. TTDSS Model Framework

Hierarchical Artificial Neural Network (HANN)

The HANN module passes through a chain of six consecutive decision-making systems that are linked through an automated system to guarantee limiting the number of decision variables, based on the nature of the inputs. This newly introduced framework simulates the real-life thinking of decision-makers while taking their decisions for the application of TTs. As shown in Figure 3, the 1st chain of hierarchies selects either method OCC or TT. If the decision is OCC, then the process ends and the OCC will be the chosen construction method to be implemented for this project. However, if TT is chosen, then the system automatically moves to the 2nd chain of hierarchies, which selects either directional or non-directional TT. If the decision is directional TT, the system automatically moves to the 3rd chain of hierarchies. But, if the decision is non-directional TT, the system automatically moves to the 4th chain of hierarchies. The 3rd chain of hierarchies contains the list of directional TT either MT or HDD. If MT is chosen, then the system automatically moves to the 5th chain of hierarchies to choose either

OMT or CMT. But, if HDD is chosen, the system automatically moves to the 6th hierarchy to choose one of these three construction methods: Mini-HDD, Mid-HDD, and Maxi-HDD. The 4th chain of hierarchies contains the list of non-directional TT varying among HDD, HAB and PR. If HDD is chosen, the system automatically moves to the 6th chain of hierarchies as discussed above.

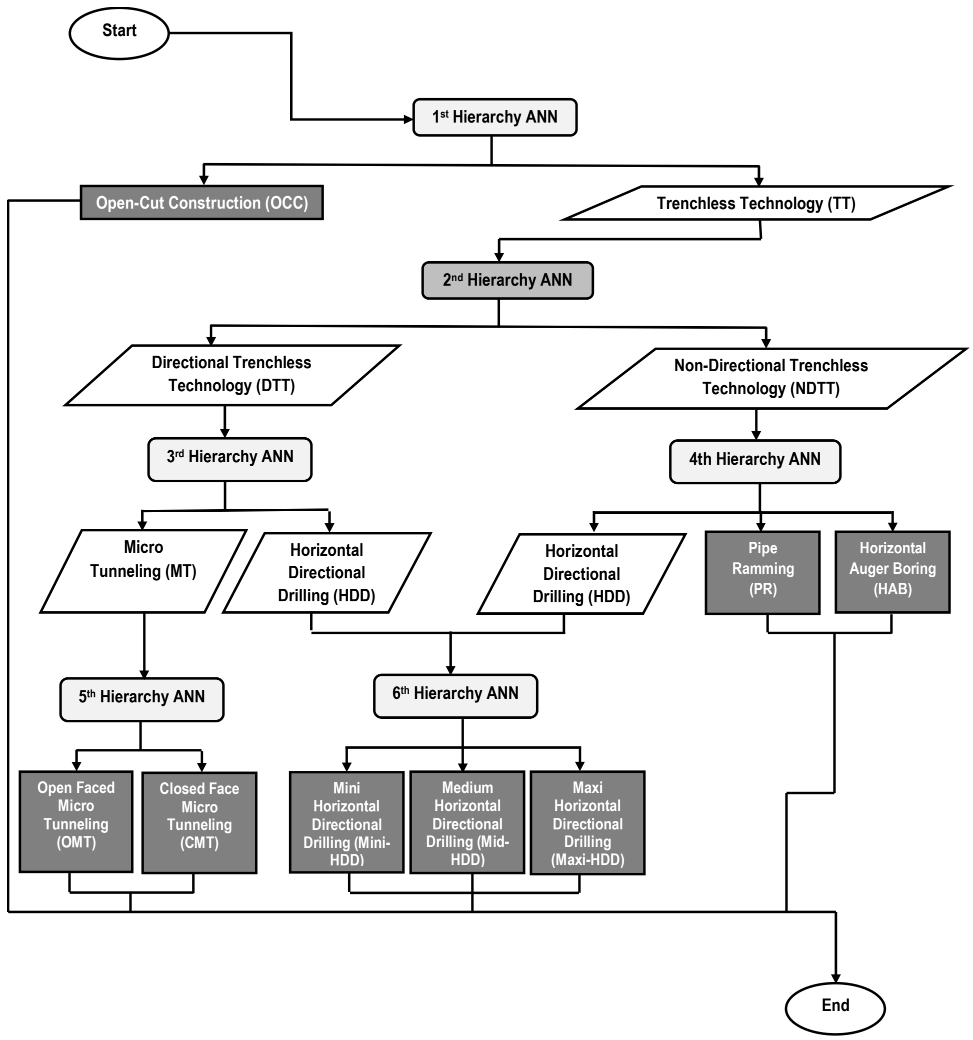

Figure 3. The Hierarchical Artificial Neural Network (HANN)

Modeling Setup

The 1st HANN obtains the required project data from the central database module. To guarantee a high precision, attributes affecting the choice between OCC and TT are defined: total project length, drive length, required accuracy, soil type, ground water impacts (e.g., de-watering), existing underground utilities, surrounding above-ground structures, work space requirements (e.g., street width), acceptable noise level, traffic impacts, pipe diameter, pipe material and pipe depth. Projects are processed and the ANN starts with the 1st level of hidden neurons to obtain the impact of each factor on the output. The 2nd level of hidden neurons chooses between OCC and TT based on the pre-defined factors. Several functions were investigated and the best fitting one, that reached the lowest training percentage of error, is the Int(Tanh) resulting in -1 and 1 representing the OCC and TT respectively.

The 2nd HANN chooses TT projects and obtains their data from the central database module. The main factors affecting the choice between DTT and NDTT are accuracy and pipe material. All TT projects are processed and the ANN begins with the 1st level of hidden neurons to obtain the effect of each factor on the output. The 2nd level of hidden neurons chooses between DTT and NDTT based on the pre-defined factors. The 3rd HANN selects DTT projects and obtains their data from the central database module. The main factors affecting the choice between MT and HDD are pipe material and type of soil. All the DTT projects are processed and the ANN starts with the 1st level of hidden neurons to obtain the effect of each factor. The 2nd level of hidden neurons chooses between MT and HDD based on the pre-defined factors. The 4th HANN selects NDTT projects and obtains their data from the central database module. The main factors that affect the choice between HAB, HDD and PR are pipe material and soil type. All the NDTT projects are processed and the ANN begins with the 1st level of hidden neurons to obtain the effect of each factor. The 2nd level of hidden neurons chooses between HAB, HDD and PR.

The 5th HANN selects MT projects and obtains their data from the central database module. The main factor affecting the choice between OMT and CMT is ground water existence. All MT projects are processed and the ANN passes through one level of hidden neurons to obtain the effect of the only factor. The 6th hierarchical selects HDD projects resulting from both hierarchies and obtains their data from the central database module. The main factors that affect the choice are drive length and pipe diameter. All MT projects are processed and the ANN starts with the 1st level of hidden neurons to obtain the effect of each factor. The 2nd level of hidden neurons chooses between Mini-HDD, Mid-HDD, and Maxi-HDD.

Optimization Engine

The Optimization Engine features the MS Excel® Evolver TM V.5.5 add-in, and uses the GA optimization option. The complex nature of the previously introduced problem initiated the need for integrating the HANN with the GA-based optimization engine for applying the concept of perquisite hierarchies with an objective of minimizing the percentage error between TTDSS and actual outputs. Table 3 shows the optimization attributes that was identified for this complex problem, defined separately for the 6 HANN's. As shown in Table 3, the objective was to minimize the percentage of error from the actual applied construction method in each hierarchy. The variables are the hidden neurons, which differ according to the number of factors considered in the hierarchy.

Table 3. Optimization Attributes

Attribute	Description
Objective function	Minimize the percentage of error between the construction methods recommended by the TTDSS vs. the actual output for the training cases.
Decision Variables	The weights among the input and hidden neurons and hidden neurons and output neurons.

Model Training and Testing

To validate the proposed approach, 60 different projects were introduced to the system. The input neurons contained the following: drive length, soil type, pipe diameter, pipe depth, project type, usage, surrounding above conditions, existing underground utilities, traffic impact, noise level, project length, ground water existence, and pipe material. The output neurons differ from one hierarchy to another. These output neurons are used as inputs for the higher level HANN to achieve the final TTDSS output (most appropriate construction method). 80% of the projects (48 projects) were taken as training cases for the system, while 20% (12 projects) were taken as testing cases. The system functions through 4 main modules as follows: (1) Central Database Module, (2) HANNs, (3) Optimization Engine, and (4) Project Construction Method Module. The central database module contained the 60 projects inputs and actual outputs. The HANN was composed of: (1) an input layer to convert the subjective factors into numerical inputs, (2) a two-level hidden layer to guarantee more precision and accuracy for the system where the number of hidden neurons differs according to the number of factors considered in this hierarchy, (3) an output layer that converts the numerical model into subjective outputs (construction methods) to compare it with the actual output based on the pre-defined experts results. After that the optimization engine runs each hierarchy solely to get the weights for the factors considered. Finally, the projects construction method module predicts the output (construction method). The results of the TTDSS were promising, showing a negligible percentage of error for both the training and testing cases. Table 4 shows the error percentage for both the training and testing cases respectively for the six-chain of hierarchies. The integration of HANN and

GAs resulted in a low error percentage not exceeding 5% for both training and testing cases.

Table 4. Training and Testing Results

Hierarchical Chain	Training Cases (% Error)	Testing Cases (% Error)
1st	1.5%	1%
2nd	1%	0.5%
3rd	1%	1%
4th	0.5%	0%
5th	0%	0%
6th	1%	0.5%
Total	5%	4%

TTDSS promising results demonstrated the success of the newly-introduced concept of HANN to effectively provide decision makers with a reliable tool that guides them to the most appropriate construction method for their projects, based on the project nature, surroundings characteristics, etc.

CONCLUSION

TTDSS is an appropriate tool for providing decision makers with the best construction method to be applied for a given pipeline project. In this case, the new concept of hierarchies was applied on the ANN model to guarantee more precise and accurate results. The TTDSS resulted in a very low percentage of error, not exceeding 5%, which ensures the success of applying the new concept of hierarchies in the ANN. Finally, the flexibility of the TTDSS widens its application for use in many locations and project environments.

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Water Pipeline from Turkey to Cyprus—1,600 mm Diameter Polyethylene 100 Pipeline and Its Flange-Technology Solution

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Abstract

The use of polyethylene (PE) pressure pipelines is increasing in international significance and therefore being used in increasingly challenging conditions. The reasons for this are manifold, including the excellent corrosion resistance of the material, the flexibility of high-density polyethylene 100 (PE 100) pipes, and the resulting installation and cost advantages. The construction of a80 km long water pipeline through the Mediterranean sets new standards in water supply. Requested by the Turkish Republic of Northern Cyprus (TRNC), it runs from the Turkish mainland (Mersin Province) to the Turkish part of the island of Cyprus. For the project, individual lengths of 500 m (1600 ft) PE 100 pipe were produced and installed so that they float 250 m (918 ft) below the surface of the Mediterranean. In 2011, a dam was built in the mountains of the province of Mersin which directs water from the Dragon River to the north-east of the town of Anamur. The dam is also to be used for generating hydroelectric power, as well as for storing the water for the pipeline. The prerequisite for the success of this project was the development of an ultra-demanding and innovative flange design for a stub end that can durably join steel and PE 100.Due to the excellent communication and cooperation between Reinert-Ritz GmbH and the project managers, a high level of confidence was generated for the long-term tightness and safety of the flange connection.

INTRODUCTION

To meet the water needs of the Turkish Republic of Northern Cyprus (TRNC), a project was developed to transfer water through a pipeline from Turkey to the TRNC. The planning phase of this project started in 1998 and was approved by the Council of Minister's Decree No. 98/11202 on May 27, 1998.Engineering services for the project have been included in the investment program of the Turkish State Hydraulic Works (DSİ) since 2002.The project involves on-shore structures as well as off-shore structures. The on-shore structures consist of the construction of a dam, storage tanks, pumping stations, valve chambers and transmission lines. The off-shore structures consist of manufacturing facilities and the construction of a pipeline at a depth of 250-280 m (820-918 ft) in the Mediterranean. Water will be carried from the Alaköprü Dam, built on the Dragon River, to Anamur in Turkey and from there through the Mediterranean to the existing Geçitköy Small Dam in TRNC.By raising the height of the Geçitköy Small Dam, it will be converted to Geçitköy Dam, a reservoir that will be used for water supply and irrigation. The pipeline will supply 75 million m³ (60,800 acre ft) of water per year from southern Turkey to the TRNC.

PROJECT COST

A $450 million intergovernmental framework agreement for the gigantic project, called the Peace River Project, was signed in July 2010. The project, planned to be completed by July 2015, is funded by Turkey. The total investment cost of the project is budgeted at TUL 782 million (approx. US$ 432 million) consisting of TUL 45.6 million (approx. US$ 25.2 million) for structures in Turkey, TUL 630 million (approx. US$ 348 million) for the undersea pipeline, and TUL 26.9 million (approx. US$ 14.9 million) for the structures in Northern Cyprus.

CONSTRUCTION STAGES

The project includes the construction of a dam and a pumping station on both sides of the project, as well as a pipeline of 107 kilometres (66 miles) that mainly runs across the sea. The construction will have five stages:

Figure 1: *Overview of Construction Stages*

Stage 1 - Alaköprü Dam on the Turkish Side

In 2011, a dam was built in the mountains of the province of Mersin which leads from the Dragon River to the north-east of the town of Anamur. The dam is also to be used for generating hydroelectric power, using a new plant with a capacity of 26 megawatts, as well as for storing the water required for the pipeline.

Overall, the water storage capacity of the dam, collected from the Dragon River, is 130.5 million m³ (106,000 acre ft). The height of the dam is 88 m (289 ft) above the river bed and 93 m (305 ft) from the base.

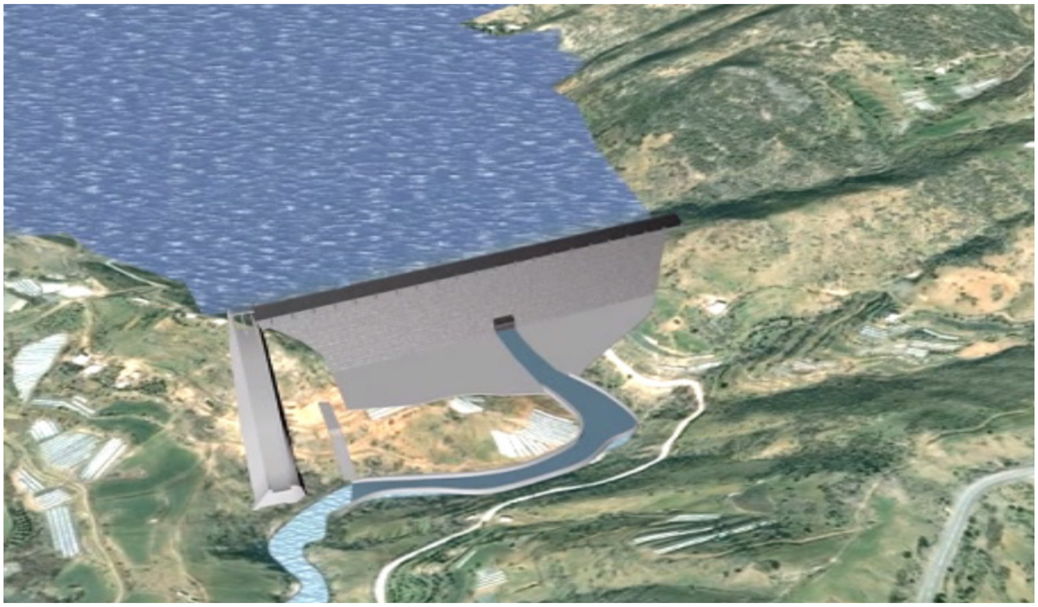

Figure 2: *Alaköprü Dam*

Stage 2 - Ductile Iron Pipe on the Turkish Side

From the dam to the coast, the water is transported by gravity. The transporting ductile iron pipe of 1,500 mm (60 in) diameter and a total length of 23 km (14.4 miles) ends in the Anamur valve chamber next to the coast.

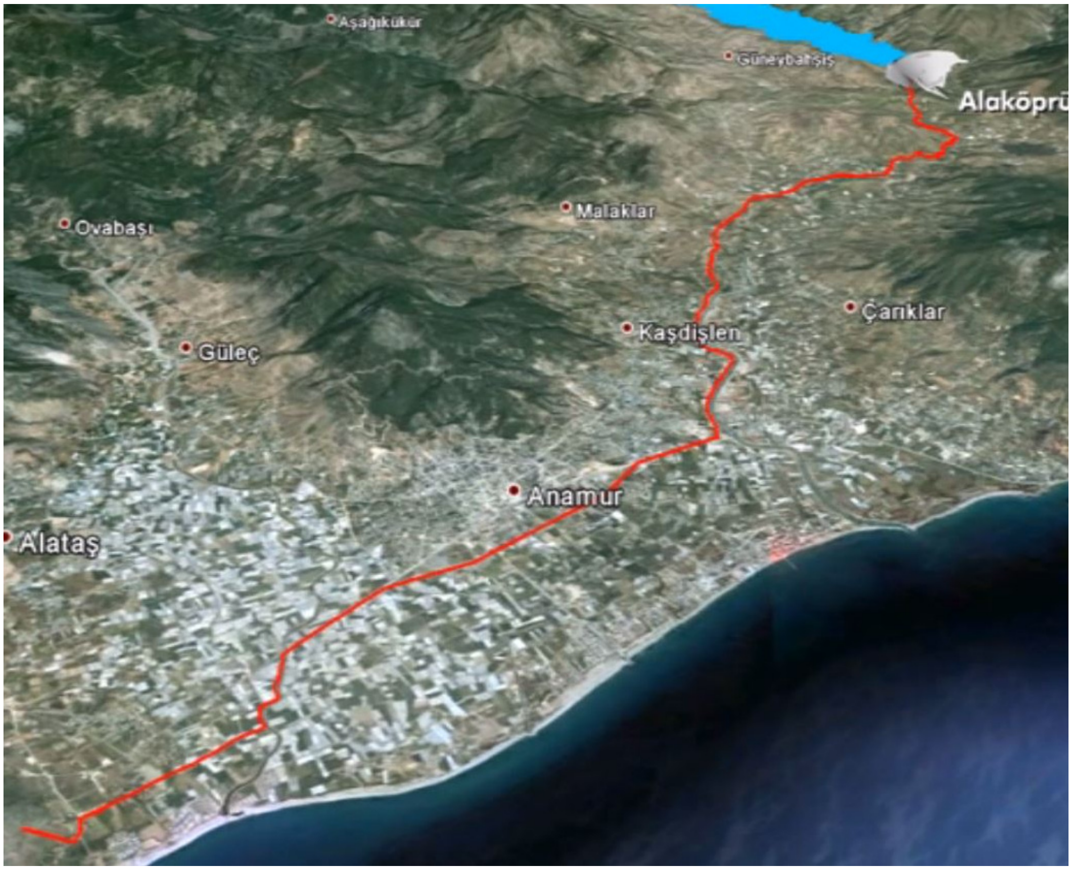

Figure 3: *Ductile Iron Pipe Path on the Turkish Side*

Stage 3 - PE 100 Sea Crossing

The construction of the 80 km (≈ 50 mile) long drinking-water pipeline through the Mediterranean Sea sets new standards in water supply and is unique in the world. The sea crossing is divided into three sections.

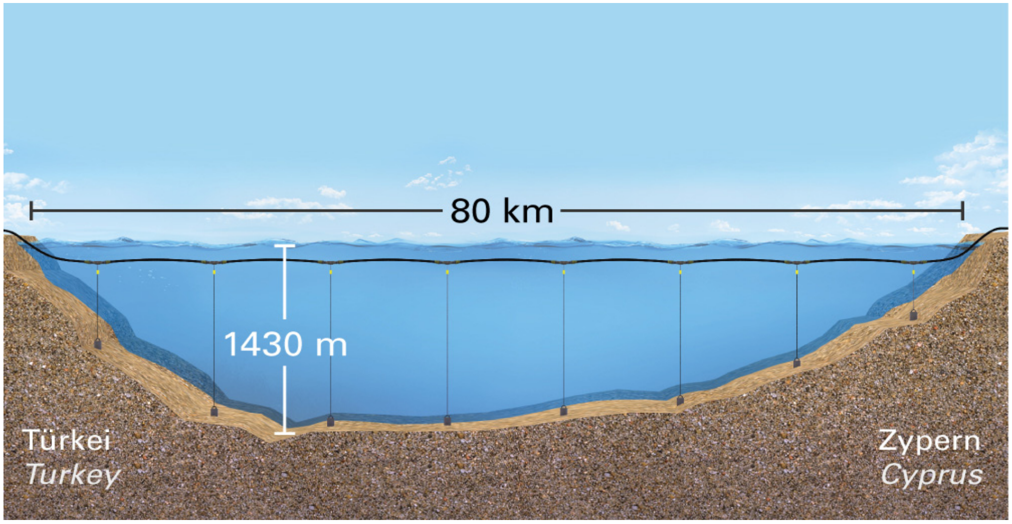

Figure 4: *Sea Crossing Overview*

Section 1 - Transition from the valve chambers to the off-shore section
From the valve chambers on the Turkey side runs a 1,600 mm diameter (63 in), SDR 21, PE 100 pipe in a water depth of 20 m (66 ft) below sea level. It is surrounded by round concrete blocks and covered with gravel to resist the effect of buoyancy.

Figure 5: *PE 100 Pipe In-Shore Transition*

Section 2 - Off-shore laying with concrete blocks
The next step is to transition from the shallower edge of the sea to reach the deeper, first anchor point from which the PE 100 pipe begins to float in the water. The PE 100 pipe in this and the following section has an SDR of 21. In this section it is fixed to the seabed with concrete blocks.

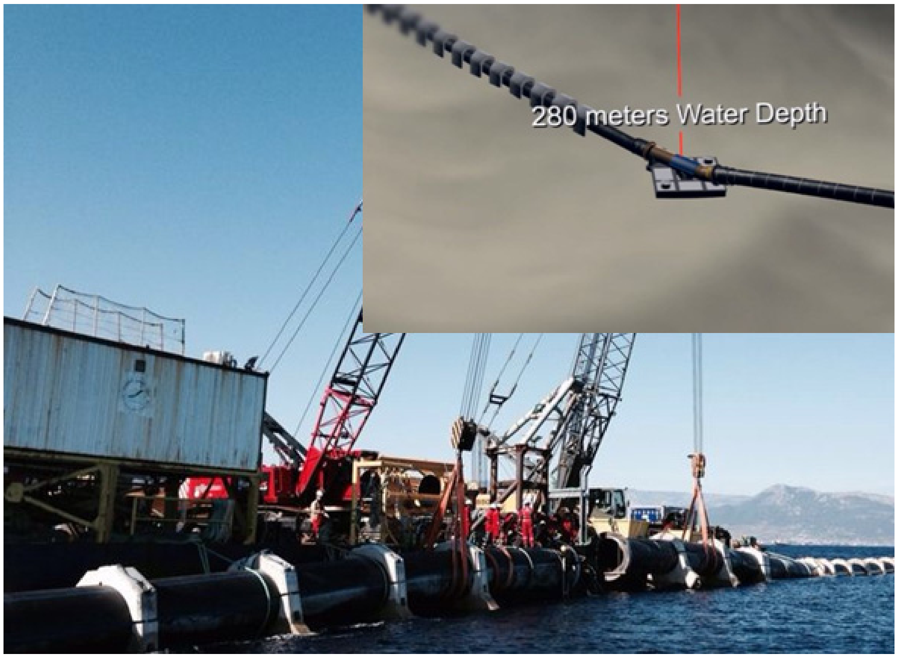

Figure 6: *PE 100 Pipe with Concrete Blocks*

Section 3 - 80 km of floating and tethered pipeline
Single lengths of 500 m (1,600 ft) PE 100 pipe are installed at a depth of 250-280 m (820-918 ft) below the surface of the Mediterranean Sea. The line is anchored every 500 m (1,600 ft) and floats in the sea.

The production of pipes for the project begins close to the Taşucu receiving basin. Here, the 500 m (1,600 ft) long sections of PE 100 pipes with an outside diameter of 1,600 mm (63 in) are manufactured by three extrusion machines in parallel. This not only achieves cost advantages, but for further projects, it provides a security factor which is desirable and mandatory in many cases.

The design pipe length results from the fact that PE 100 pipes, due to their specific gravity of less than 1 g/cm³ (≈0.955 g/cc), would normally float on the surface of the water. This fact, in combination with the lower relative density of freshwater compared to seawater, causes enormous buoyancy forces. These buoyancy forces are counteracted by the pipe joining yokes, each consisting of two flanged connectors and a steel pipe bend to which the flanged connector is fixed. The steel yokes have an outside diameter of 1,514 mm (≈60 in), a curvature radius of 8,000 mm (≈315 in) with a bend angle of 30°, and weigh around 10 tonnes (11 tons) without the connecting elements. The total weight, including the two connecting elements, is around 13 tonnes (14.33 tons). The steels yokes, which also act as fixing and anchoring points, are subsequently drawn down to a depth of 280 m (820 ft) by means of steel cables and then anchored to the seabed, illustrating even better the special features of this project.

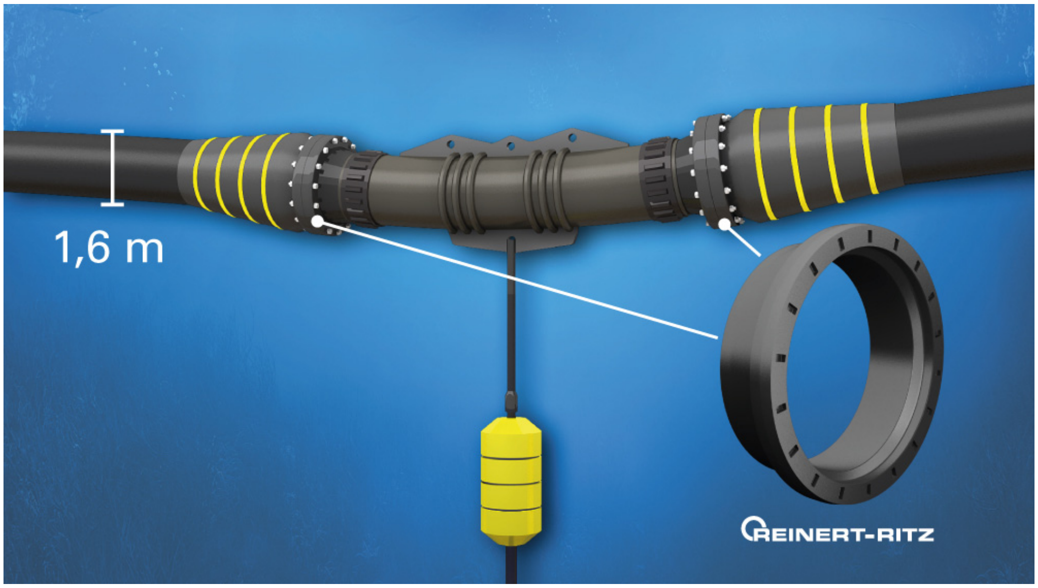

Figure 7: *Yoke Point Every 500 m (1,600 ft)*

The otherwise customary installation was not the first-choice solution for this project. One of the reasons was the depth exceeding 1,430 m (4,690 ft), interspersed with numerous underwater ridges and trenches along the installation route. The installation of the PE 100 pressure pipeline is now merely the conclusion of a long chain of events.

Stage 4 - Force Main on Cyprus Side

From the Cyprus coast, the water is transported by a 1,400 mm (55 in) force main pipeline made of ductile iron to the Güzelyali Pumping Station where it is pumped into the reservoir of the Geçitköy Dam, 3,157 meters (≈ 2 miles) away.

Figure 8: *Güzelyali Pumping Station*

Stage 5 - Geçitköy Dam on Cypriot Side

Geçitköy Dam, which has a height of 65 m (213 ft) from the ground and 58 m (190 ft) from the river bed, has a storage capacity of 26.5 million m³ (21,500 acre ft).

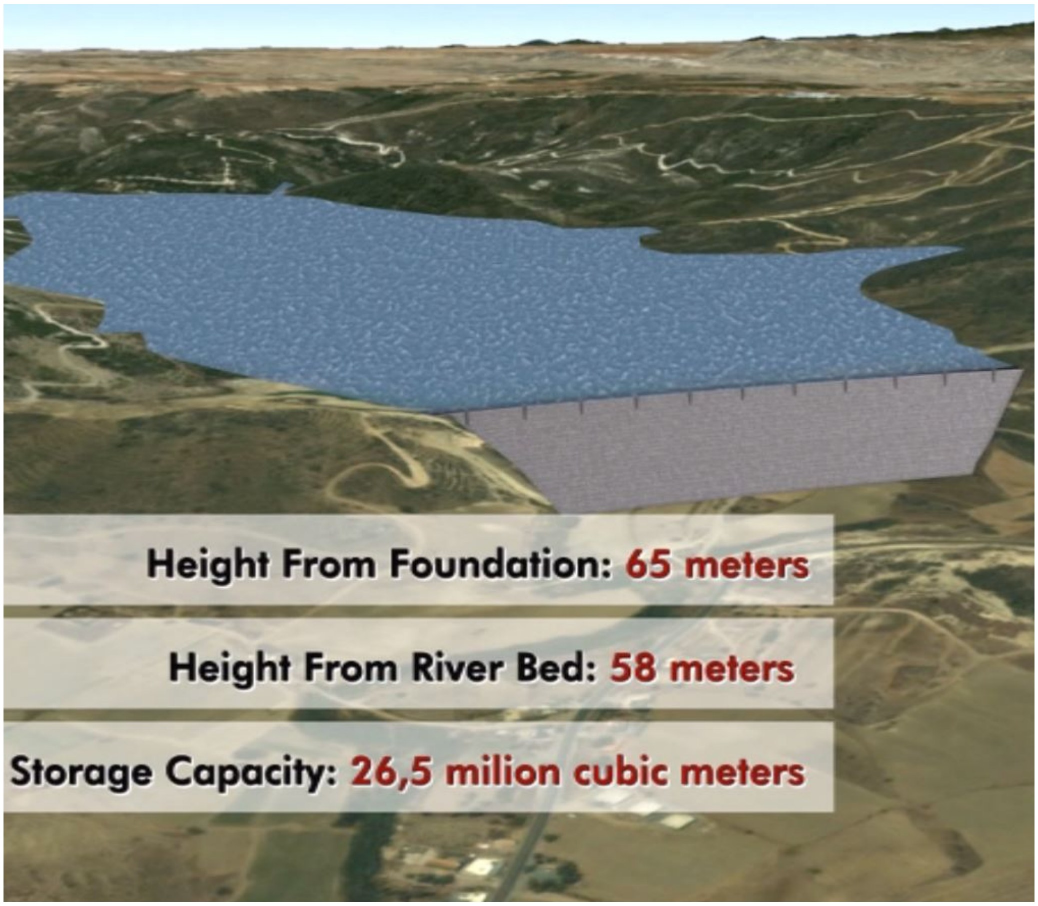

Figure 9: *Geçitköy Dam on Cyprus Side*

DEVELOPMENT OF THE FLANGE CONNECTION

On site, the custom made stub ends are welded to the PE pipes and bolted to the steel tube yokes by means of a special flange provided for this purpose. This yoke is then anchored to the seabed by means of steel cables to the securing and anchoring point. The challenge for Reinert-Ritz was to provide a product to join the PE pipes and steel yokes durably and securely so that even the most adverse conditions encountered underwater in the Mediterranean Sea could not impair the connection. At a depth of about 250 m (820 ft), the design has to deal with the following technical challenges: buoyancy forces exerted by the PE 100 pipe, stress due to buoyancy caused by conveying drinking water in a salt-water environment, powerful and dynamic sea currents, high marine and submarine traffic frequency, potential earthquakes and the operating pressure of 7 bar (101 psi). The encapsulated flange connection is additionally supported by a polyurethane stiffening system. The expected forces were simulated in an approved testing facility.

Figure 10: *Test for the Whole Flange Connection*

The extreme dynamic loads to which the flanged connection is exposed in underwater operation were considered during the development.

The extrusion of hollow bars of 1,900 mm (75 in) / 1,400 mm (55 in) diameter with a wall thickness of approx. 250 mm (10 in) is the first stage in the fabrication of the special flanged connection. The subsequent machining takes place on a milling machine capable of processing material up to sizes of 2,800 x 1,500 x 4,800 mm.

Figure 11: *Milling process at Reinert-Ritz*

A particular challenge was presented by the large dimensions of the PE pipeline and the extremely high dimensional accuracy required, with tolerances of -0.0 mm and +0.5 mm (-0.0 in and +0.002 in). On site in Turkey, the finished PE 100 stub end is secured geometrically in two steel clamping assemblies into which it must fit perfectly.

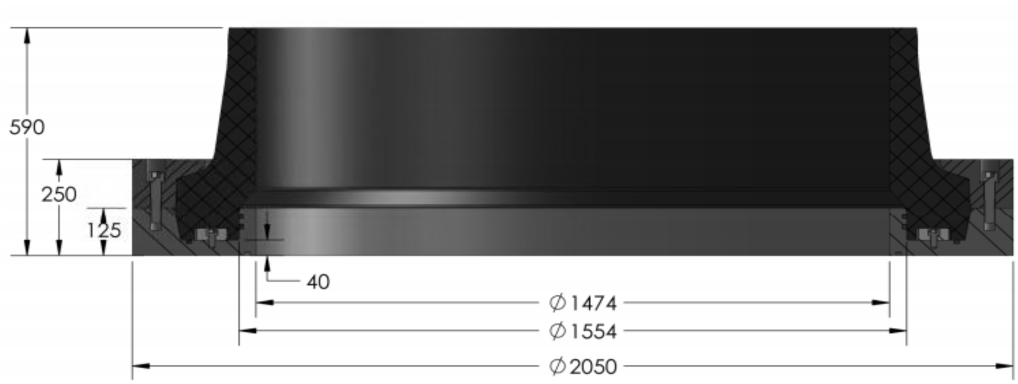

Figure 12: *Encapsulated PE 100 Stub End Cross Section*

On a special installation ship, the high-precision flanged connectors are assembled on both ends of the steel yoke with twenty-four connecting bolts. The yoke is then anchored to the seabed, in order to enable the pressure pipeline to float at an average depth of 250-280 m (820-918 ft).

DECISION FOR HIGH QUALITY AND SAFETY

Reinert-Ritz was able to meet the project's requirements of full pressure classification, according to the nominal pressure of the pipe, for large dimension fittings up to an outside diameter of 2,000 mm (79 in). Over forty years of experience in plastics as well as the company's pronounced standards for quality, helped it to become a valuable member of the project team. From the high quality of the resin used for the production of the semi-finished products, through to the machining of the finished stub ends, it was able to guarantee the high production standards vital for the performance and completion of this demanding project. There are different production routes for full pressure rated fittings, for example, injection moulding and machining. The hollow bars and solid rods horizontally extruded are used in the latter process. This method provides a product free of voids that reflects their many years of design processing and testing experience.

CONCLUSION

The prerequisite for the success of this project was the successful development of an ultra-demanding and innovative flange design for a stub end that durably joins steel and PE 100. The extreme dynamic loads to which the flanged connection is exposed in underwater operation were considered during the development. Due to the excellent communication and cooperation between Reinert-Ritz GmbH and the project managers, a high level of confidence was generated for the long-term tightness and safety of the flange connection.

Make Way for Progress—The Challenges of Relocating Large Diameter Water Mains for Light Rail System Expansion

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Abstract

The Valley Transit Authority is working with the Bay Area Rapid Transit (BART) to ultimately expand BART's existing light rail system south to San Jose. Several new stations are proposed as part of the Light Rail Track extension. However, two of the new stations will impact two existing large diameter water pipelines, owned by the Santa Clara Valley Water District. These existing lines have to be relocated away from the new stations. This paper will discuss the technical design aspects of installing new pipe and fittings into an existing 66" concrete pressure pipe raw water main and into an existing 42" steel pipe treated water main in order to properly relocate both lines. A key consideration in this modification to the existing lines is the thrust forces generated from the realignment. The thrust restraint of the old 66" line posed a number of challenges since the original pipe in this area did not have longitudinal thrust forces. Additionally the owner, the Santa Clara Valley Water District, has their own specialized thrust restraint procedure that is based on a 1960s paper. This approach is different than what is recommended by the AWWA in the pertinent AWWA design manuals. Other challenges encountered during the relocation design included a number of pipe design considerations such as internal and external load, buckling, and fittings design. Further complicating the relocation effort is the need to account for a special "rattle box" casing to protect existing gas lines under a new "floating slab" for the Light Rail Track at one location. Included in the discussion will be recommendations on a few "lessons learned" from the relocation design efforts.

BACKGROUND

Big changes are coming to Silicon Valley with the San Francisco 49er's football team building a new stadium in San Jose, leaving their friendly confines in San Francisco. With this new stadium located further south than the southernmost station on the

BART rail system, along with the influx of people that have moved to Silicon Valley over the past few decades, the time was ripe for BART to extend their light rail system south.

The project was awarded as a Design-Build contract to the joint-venture construction team of Skanska Shimmick Herzog (SSH) who teamed with Lockwood, Andrews and Newnam, Inc. (LAN) and TY Lin International as their project engineers.

Several new stations would have to be constructed along the 10 additional miles of this rail extension, and it was found that there were existing water pipelines owned by the Santa Clara Valley Water District (SCVWD) in the way. It was quickly decided by the Design-Build team that it would be easier to "simply" relocate these water lines rather than finding other sites for the new stations. A local San Jose firm, HMH Engineers (HMH), was named as the lead engineers for this relocation effort, who in turned asked LAN to assist with the large diameter design issues. HMH would provide the pipeline design and agency interface while LAN would provide the pipe design calculations.

A number of factors made this project more challenging than expected. First, the SCVWD has very limited experience with design-build projects on SCVWD facilities. With the critical nature of the SCVWD pipelines to the county's water supply, the cut-over times to tie-in both relocated pipelines to the existing pipelines were limited to 10 calendar days each, and there were very limited tie-in windows. Finally, due to the size of the lines and their critical nature, SCVWD's pipelines are not often relocated and the agency demands a <u>very</u> cautious, robust, and long time-horizon approach to relocation.

Existing Water Lines

The two existing SCVWD pipelines to have sections relocated were the 66-inch Central Pipeline, constructed in 1964-5, and the 42-inch Milpitas Pipeline constructed in 1992. Fortunately for the design team, SCVWD keeps excellent records and was able to provide HMH/LAN with original plans, line layouts and as-built drawings for these two pipelines.

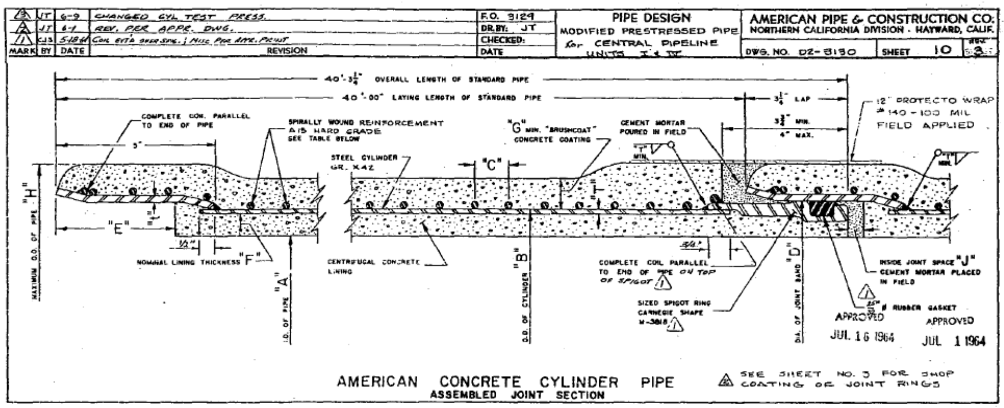
The first challenge was determining the existing pipe material utilized on the 66-inch Central Pipeline in the relocation area, as it appeared three different pipe materials might have been provided for this line according to the historical information. These three materials are:

- Embedded Cylinder Prestressed Concrete Cylinder Pipe (PCCP) designed in accordance with the original project specifications and the AWWA C301 standard that was current at that time.
- Modified Prestressed Pipe (also referred to as Concrete Cylinder Pipe (CCP)) that American Pipe and Construction (now NOV-Ameron) designed in accordance with the original project specifications and Federal Specification

SS-P-381A. Today we refer to this product as Concrete Pressure Pipe, Bar-Wrapped, Steel-Cylinder Type per AWWA C303.

- Welded Steel Pipe with cement mortar lining and cement mortar coating and a rod wrap over the steel cylinder. This was a hybrid product, part steel pipe per API 5LX with bar wrapping, cement mortar lining and cement mortar coating similar to SS-P-381A.

Upon closer examination of the pipeline documentation, the pipe material product that the design team would have to work with was the second bulleted item above, CCP. Figure 1 presents the joint cross section and the design of the existing pipe.



MANUFACTURING DATA																				INSTALLATION DATA					
						STEEL CYLINDER				REINFORCING ROD															
INSIDE DIAM. OF PIPE A (IN.)	SPECIFIED PIPE CLASS	PIPE DESIGN PRESSURE (WD)	APPR. FOOTAGE	PLANT TEST PRESSURE COMPL. PIPE	TOTAL A_T/FT. (SQ.IN.)	OUTSIDE DIAMETER B (IN.)	GAGE OR THICKNESS T (NO. OR IN.)	A_Y/FT. (SQ. IN.)	PLANT TEST PRESSURE (P.S.I.)	A_S/FT. (SQ.IN.)	COILS PER FOOT (NO.)	ROD DIAM. (IN.)	SPACING C TO C C (IN.)	NOMINAL JOINT DIAMETER D (IN.)	BELL RING THICKNESS t (IN.)	DEPTH OF SOCKET E (IN.)	NOMINAL LINING THICKNESS F (IN.)	COATING THICKNESS G (IN.)	MAXIMUM OUTSIDE DIAMETER OF PIPE H (IN.)	INSIDE JOINT SPACE J (IN.)	MAXIMUM PULL PER JOINT (ANGLE)	MINIMUM PULLED RADIUS USING 40' PIPE (FEET)	OFFSET IN 40' FOR MAXIMUM PULL (FEET)	APPROX. WEIGHT OF 40' PIPE (POUNDS)
66"		300		195	2.88	67 7/8"	0.194"	2.33	165	0.55"	11.28	1/4	1.07"	68 3/8	5/16"	3 3/4"	5/8" ± 1/8"	1 1/4"	71 1/2"	1/2"	0°50'	2750	0.59'	24,600
		325		211	3.00		0.194"	2.33	165	0.67	8.72	5/16	1.33"		5/16"			1 5/16"	71 5/8"					25,400
		350		228	3.23		0.194"	2.33	165	0.90	8.14	3/8	1.48"		5/16"			1 3/8"	71 3/4"					27,200
		375		244	3.46		0.219"	2.63	186	0.84	10.90	5/16	1.10"		5/16"			1 5/16"	71 5/8					27,100
		400		260	3.69		0.219"	2.63	186	1.07	9.65	3/8	1.24"		3/8"			1 3/8"	71 7/8					28,300
		425		276	3.92		0.234"	2.81	198	1.11	10.02	3/8	1.20"		3/8"			1 3/8"	71 7/8					28,800
		450		292	4.15		0.234"	2.81	198	1.34	6.81	1/2	1.76"		3/8"			1 1/2"	72 1/8					30,600
		475		308	4.38		0.234"	2.81	198	1.57	7.99	1/2	1.50"		7/16"			1 1/2"	72 1/4					31,200
		500		325	4.62		0.250"	3.00	212	1.62	8.24	1/2	1.46"		7/16"			1 1/2"	72 1/4					31,700
		525		341	4.85		0.250"	3.00	212	1.86	9.47	1/2	1.27"		7/16"			1 1/2"	72 1/4					32,300

Figure 1 – 66-inch CCP

The line layout drawings for the 66-inch CCP showed the pipeline areas that contained restrained joints for thrust restraint along with areas where push-on, rubber gasketed, Carnegie-style joints were utilized. In the area of the proposed relocation, only the rubber gasketed 66-inch CCP was installed, meaning that the relocation design would have to address thrust issues that would arise from the relocation.

SCVWD's records for the 42-inch Milpitas Pipeline clearly indicated this was constructed with cement mortar lined and tape coated steel pipe as shown in Figure 2.

As can easily be seen in Figure 2, the existing 42-inch diameter pipe is operating at high pressure. Furthermore, as has been SCVWD standard design philosophy for reliability and protection of the public water supply, the field joints were all bell and spigot welded lap joints. Because a fully welded steel pipeline provides restraint against movement due to thrust forces, the relocation of this 42-inch pipe was thought to be much less of a design problem.

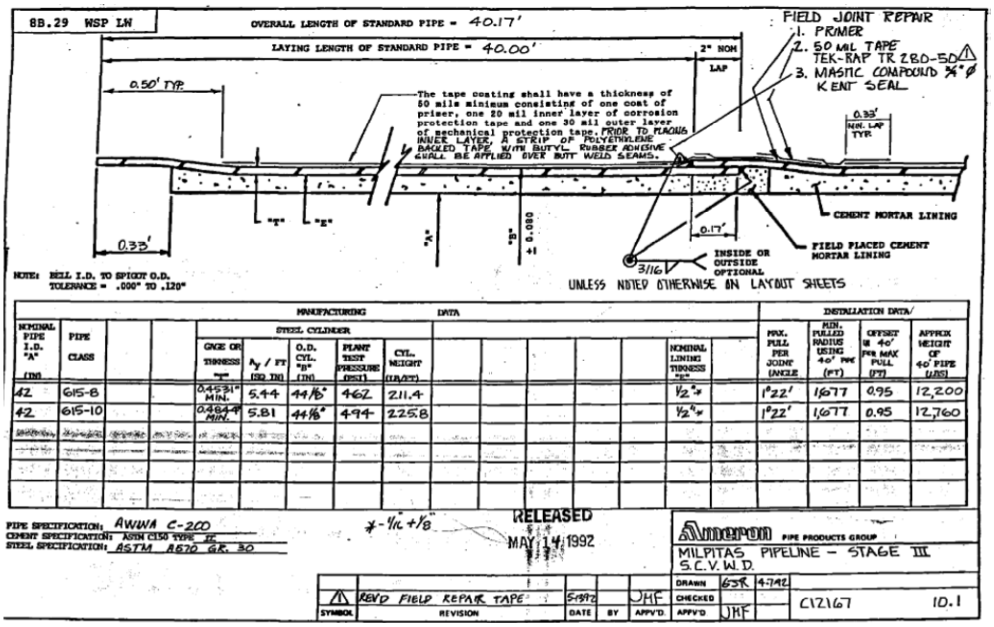


Figure 2 – 42-inch Steel Pipe

Relocated Water Lines

SCVWD required that the new relocated pipe be steel pipe and not PCCP or CCP. This was felt to be the best option to minimize future maintenance and risk to adjacent facilities and to water pipeline function. Furthermore, all field joints were specified to be welded, utilizing bell and spigot lap joints. Because elbows would be required to realign the existing pipe to accommodate construction of the stations, it was decided that the design and pipe installation would be much “simpler” and “cleaner” with all welded restrained joints. Finally, SCVWD required that the steel pipe have the same type of coating as the existing pipe. This would simplify corrosion control and minimize corrosion risks to both the new and old pipe. Therefore, the 66-inch pipe would have cement mortar coating and the 42-inch pipe would have tape coating.

DESIGN CHALLENGES

Design Challenge One – Thrust Restraint

Thrust restraint for steel pipe is typically accomplished in accordance with AWWA M11 along with any owner supplemental design requirements. However thrust restraint for CCP is now often accomplished utilizing AWWA M9, third edition for concrete pressure pipe. This relatively new (2008) design procedure is slowly gaining acceptance with designers and owners in the United States. However, many designers and owners prefer to utilize the older, second edition of M9 (1995) as it is a much simpler design procedure.

SCVWD has their own unique thrust restraint design procedure that is based on a relatively unknown paper written in 1963 by James M. Gere, a professor at Stanford University at that time. The overall premise of the paper resembles the current M9 design procedure; that is, it utilizes frictional forces along the axis of the pipe as well as lateral displacement due to the angular change at a bend. This lateral displacement is greatest at the point of intersection of the elbow and diminishes as the length of pipe from the elbow increases. Figure 3 is from the original Gere paper, showing the length of pipe that undergoes lateral displacement, denoted as bL. Where the pipe undergoes both lateral and longitudinal displacement (the bL length), the cylinder thickness in concrete pressure pipe must often be increased to withstand these combined stresses. The total "tension anchorage" length for longitudinal displacement is denoted as L.

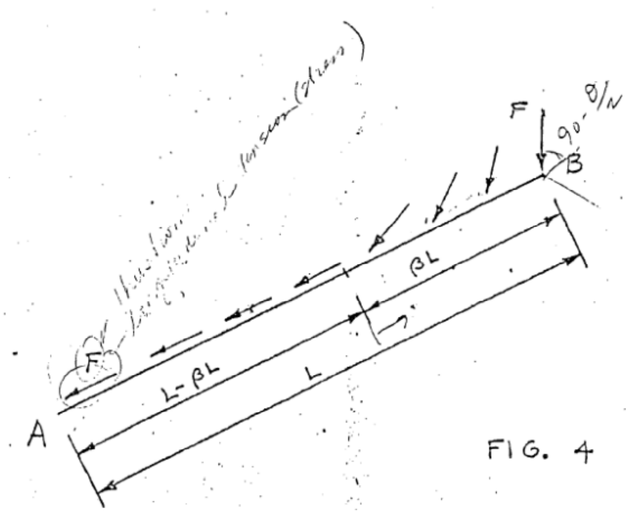


Figure 3 – Tension Anchorage Length

There are a few inherent flaws in the Gere paper that SCVWD has addressed in their own design manual, portions of which were shared with the design team. However, when the deflection angle is small, such as 30° or less, the calculated lateral displacement length (bL) may be longer than the total tension anchorage length (L) (based on the initial guess of b=1 when following the design procedure). When this happens, the SCVWD procedure is to artificially increase L by 1.55.

The thrust design of the 42-inch steel pipe was straight forward. First the design was accomplished in accordance with AWWA M11. A second design was then performed in accordance with AWWA M9 third edition, in order to combine stresses in the pipe, similar to the Gere methodology. Following is the technical memo conclusion written for the 42-inch thrust design on this project:

"*The tension zones should be designated based on Method 2 lengths. This method addresses the additive effect of bending and shear to the axial loads on the pipe, which the District is requiring. However, because every joint is still a single welded lap joint, there is no impact to the overall project.*"

The thrust restraint design for the existing 66-inch CCP required a different process. CCP has a much thinner cylinder than steel pipe, as the bar wrap accounts for much of the total steel area required in the hoop direction. However, the bar wrap steel area does not participate in resisting the longitudinal forces related to thrust restraint. Preliminary designs demonstrated that the steel cylinder and the joint ring attachment to the steel cylinder in the old existing 66-inch CCP were adequate for the thrust force generated by the 22.5° elbows involved for the relocation (see Figure 4 for new layout). Initial designs required installation of 45° elbows, but both SCVWD and the designers agreed that a smaller angle would lessen the thrust forces.

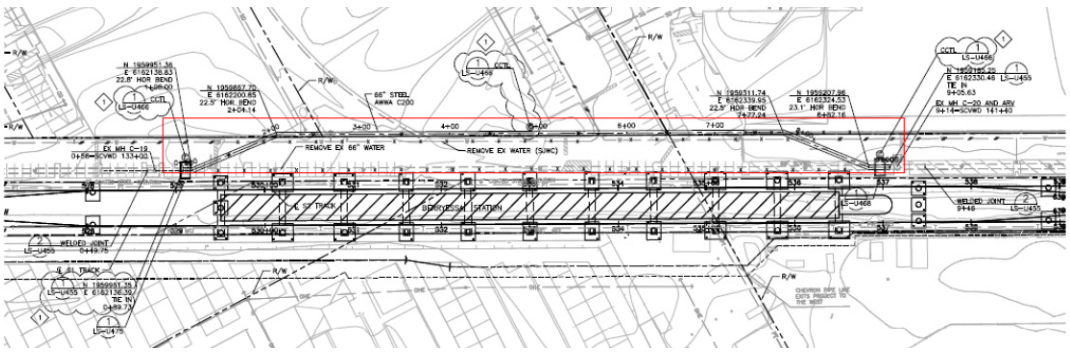


Figure 4 – 66-inch CCP Relocation

To be conservative, LAN decided to review the thrust restraint requirements for the CCP based on the following three methods:

- Method 1, AWWA M9, 2nd Edition
- Method 2, Gere as modified by the SCVWD Design Manual
- Method 3, AWWA M9, 3rd Edition

Method 1 resulted in a restrained joint length of 53 feet on each side of the elbow point of intersection (PI). Method 2 required restrained joint lengths of 59 feet on each side of the PI, but only after the original L was multiplied by 1.55 (as previously discussed). Method 3 resulted in a restrained joint length of 51 feet on each side of the PI. Similar to the 42-inch design, a technical memorandum was written for the 66-inch thrust design, with the conclusion as follows:

"*It appears that Method 3, per the current AWWA M9 manual is the proper thrust methodology to follow for designing this project. This method properly incorporates small angles into the design procedure while the Gere methodology is limited. In addition, the latest version of M9 addresses the bending and shear stresses along with soil properties in the design where the older version (2nd edition) of M9 is silent on these subjects.*

The new steel pipe 22.5° elbows may be supplied as part of a longer length of pipe (as compared to elbows for bar wrapped steel cylinder pipe or PCCP). If the "tie in" leg is longer than 16 feet from the elbow point of intersection (PI), then no other old bar wrapped steel cylinder pipe joints would need to be field welded as the restrained length would be 16+ feet due to the elbow leg length plus 40 feet for the first piece of bar wrapped steel cylinder pipe totaling the required 56 feet. If the new elbow leg *length is less than 16 feet, one additional old bar wrapped steel cylinder joint would need to be welded.*"

Concurrent with the design team's efforts, SCVWD performed their own calculations and arrived at a length of 130 feet. LAN then tried to match the 130 foot length by using differing assumptions. The closest LAN result was a length of 112 feet assuming a high ground water table. However, the original pipelines and the current soils report did not show or reflect a ground water table above the pipe. Practically, the differences in thrust restraint lengths between the two designs resulted in the need to weld only a couple of joints of the existing 66-inch CCP on either end of the relocation. Knowing that further calculation exchanges between the design team and SCVWD would only delay the pipe approval process, and considering this pipe relocation project was on the critical path for the overall BART extension, the design-build team decided to use the SCVWD calculated 130 foot length on the existing 66-inch pipe.

Design Challenge Two – Pipe and Fittings Design

Overlapping with the thrust calculations were typical pipe design calculations that included:

- Internal Pressure Design
- External Load Design Review including rail road loads
- Buckling Load Design Review
- Outlet Reinforcement Design
- Test Head Design
- Differential Settlement Design (at buried structure penetrations)

Most of the pipe and fitting designs were straight forward with minor discussion on the various pressure situations (operating, transient and field test), allowable stresses for the two pipe coating options, and steel cylinder materials and calculation procedure (AWWA or ASME Boiler and Pressure Vessel Code).

One of the major design challenges was in reaching agreement on the steel wall thickness for the 42-inch pipe. Based on internal pressure, external loads and buckling, and the steel cylinder material selected, the required design thickness was found to be 0.327-inch. However, SCVWD had originally specified a thickness of 0.4531-inch in 1992 (as shown in Figure 2). A previous relocation of a segment of this pipe under a different construction contract had utilized 42-inch steel pipe with a ½-inch steel cylinder thickness. Therefore, SCVWD required this relocation to utilize the same ½-inch thickness to achieve matching functionality, even though the thickness was not required by the design standards utilized.

66-inch pipe wall thickness design occurred without debate, as there was no adjacent/existing steel pipe thickness to match in terms of functionality. The designs reflected a steel wall thickness of 0.424-inch to satisfy all of the design requirements which was accepted by SCVWD.

Design Challenge Three – 66-inch Valve Isolation Joint Vaults
SCVWD informed the design team that recent SCVWD evaluation of pipeline corrosion control system operation showed failures (lack of isolation) in buried flanged insulating joints occurring at an unacceptable rate and frequency. With the existing old 66-inch CCP connecting to new 66-inch steel pipe, SCVWD required electrical isolation between the two pipe material products from different eras. Considering how close the connection points are to 22.5° elbows (shown on Figure 4), SCVWD also wanted to ensure bending in the flange (and adjacent flanged coupling adapter) due to the line of action of the thrust force would be handled. Finally, SCVWD required a change in how flanged isolation joints would be protected from deterioration, suggesting that placing them in vaults would be one acceptable option.

The design team designed the vault wall to take the thrust in both the vertical and horizontal directions (see Figure 5 below); therefore, the flanged insulating joint and FCA would not have any thrust forces acting on it. The pipe to vault wall connection would need to be analyzed to ensure adequate transfer of forces without damage to the pipe. Additionally, the pipe outside of the vault would need to be checked against a shear failure due to differential settlement.

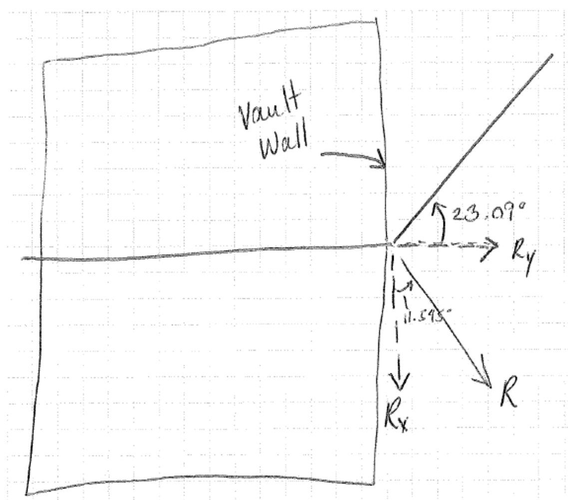


Figure 5 – LAN Schematic Drawing

The back and forth discussions and calculations on this issue occurred before final agreement could be reached. (Final design acceptance was reached the day before the start of the 10-day shut down for the tie-in.) In the end, two isolation joint vaults as depicted in Figure 6 below were included in the relocation project.

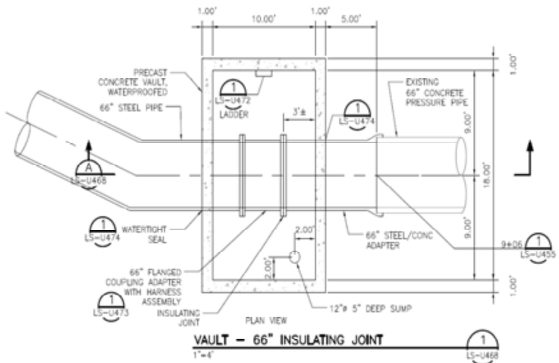


Figure 6 – 66-inch Isolation Joint Vault

Design Challenge Four – "Rattle Box"
The final major design challenge did not involve SCVWD but rather Chevron and BART. There was an existing 8-inch diameter gas line with its 12-inch diameter casing owned by Chevron that was going to be unacceptably close to one of the new BART tracks. See Figures 7 and 8 below.

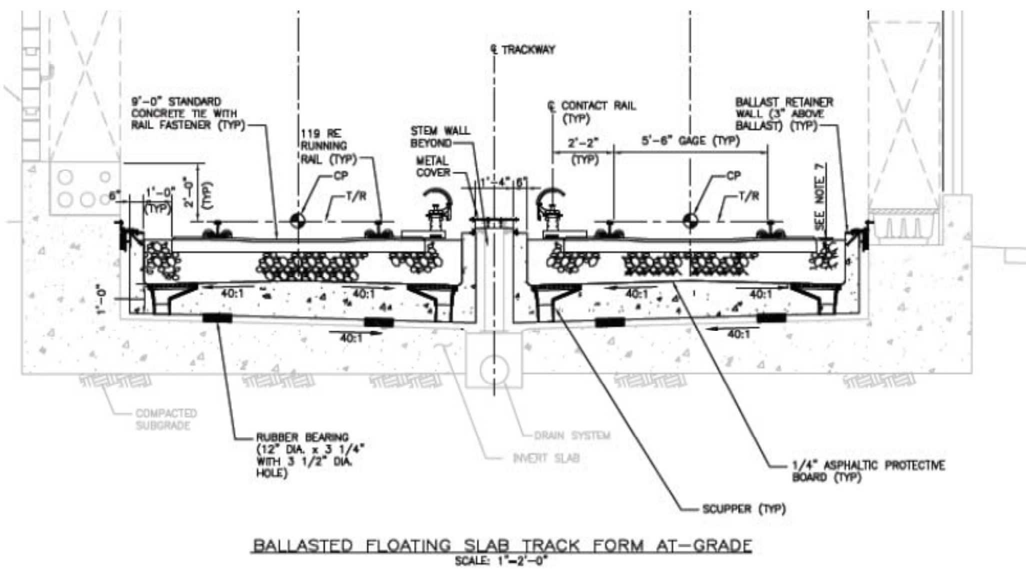


Figure 7 – Floating Slab

Chevron informed the design team that they would not allow encasement of the gas pipe and it's casing in concrete for fear of differential settlement outside of the encasement. They were very concerned about the loadings of the floating slab onto the pipe/casing as there would only be minimum cover.

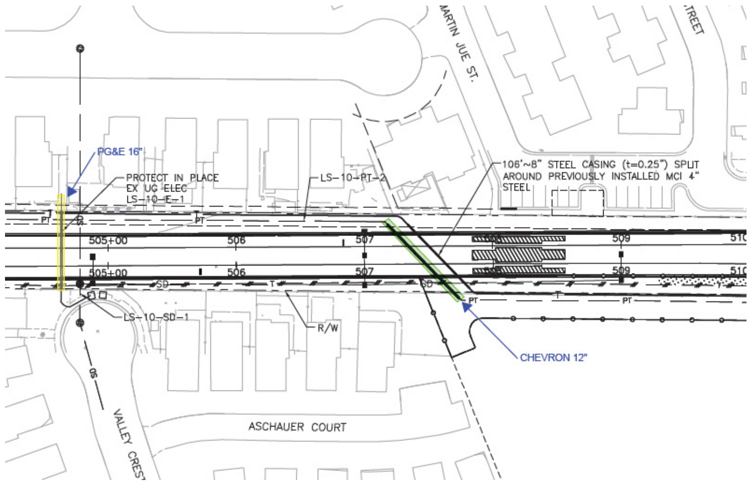


Figure 8 – Chevron 8-inch/12-inch Casing Plan

Chevron suggested a "rattle box" to protect the gas pipeline. Figure 9 below depicts the 8-inch line in a 12-inch casing protected by the rattle box developed by the designers. The rattle box looks like a larger half casing above the pipe/casing.

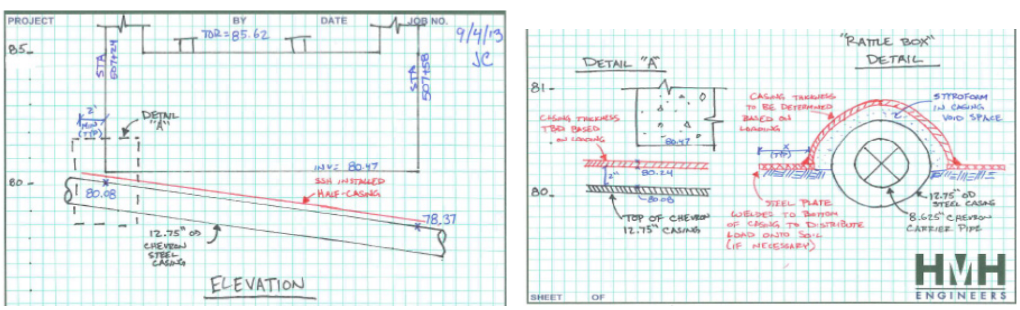


Figure 9 – 8-inch Chevron Line with Casing and Rattle Box

The calculations performed including dead and rail live loads on the 12.75-inch outside diameter casing, assuming the absence of the rattle box. The design concept for the rattle box was to provide an extra measure of safety factor against the loads.

LESSONS LEARNED

There were many lessons learned from this Design-Build pipe relocation effort. Among the chief lessons are:

- When performing engineering on design build projects, make sure the design team has all of the agencies specifications and/or design manuals. Finding out about agency design standards and manuals after submitting designs for approval causes delays and wasted effort.
- The use of design-build and the compressed design and construction processes did result in relocation of the two pipelines in much less time than SCVWD normally sees. Furthermore, this accelerated process did impact the quality of

the contract documents with more issues that needed to be resolved during construction, and not all to what SCVWD would have wanted.

- Contractor staff turnover and lack of experience in large diameter water pipeline construction resulted in a slower effort with more work by SCVWD and VTA to get results and contract compliance than should have been needed.
- Addition of an experienced pipeline design and construction subject matter expert to serve as VTA/SCVWD liaison on the pipeline relocations improved the design and construction process and helped ensure the best possible schedule outcome.
- Slower than hoped for PG&E high pressure gas line relocation (by PG&E contractor, not SSH) schedule and SCVWD windows for pipeline shutdown led to flipping of design and construction sequences for the pipelines. Although this change was not desirable, this nimbleness is part of the value of the design-build approach.
- Change in design-builder's engineering firm for pipeline relocation during selection process appear to have resulted in core large diameter water pipeline design competencies being more spread out for design-build team, and may have impacted effectiveness of design process.
- New information on the poor performance of SCVWD buried isolation joints led to substantial change in scope from what SSH bid on.
- Maintaining a margin of error during critical operations was essential. The initial SSH schedule for one of the shutdown/tie-in periods was for 10 days, ending the last day allowable. SCVWD accepted this but required starting earlier to provide a 2 day margin of error. During the tie-in/shutdown period, a SCVWD buried service butterfly valve failed in the closed position, resulting in a 2 day delay in completion.

Under the River and through the Woods: Design and Construction of Two Large Diameter Horizontal Directional Drills for the City of Corpus Christi

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Abstract

In 2009, the City of Corpus Christi (the City) moved forward with the preliminary design phase of the Mary Rhodes Phase 2 Pipeline. This 42 mile, 54-inch pipeline is integral to bringing water from the Colorado River to Lake Texana and eventually to the City's citizens. Because the City utilized a Water Infrastructure Fund deferred loan, the engineering feasibility report and environmental permitting were completed prior to the beginning of the final design phase. Environmental permits were secured in December of 2012 and final design began immediately after. With the USACE permitting conditions, trenchless construction methods were required for two large Waters of the US crossings, the Navidad River in Jackson County and the Tres Palacios River in Matagorda County. Horizontal Directional Drilling (HDD) was selected for these crossings due to the reliable accuracy, ability to drill below the water table and through wet soils, quick construction timeline, and minimal environmental impact. Multiple factors are considered in horizontal directional drilling design. Some of the key design parameters include entry and exit angles, analyzing soil conditions, and ensuring minimum depth below the river bed. The pipe material, pipe lining and coating, and pipe size are all critical components to how the HDD will be designed and constructed. The Mary Rhodes Phase 2 Pipeline HDDs were unique design challenges due to the large diameter of the pipeline, limited pipe materials available, the depth and length required, and limited working area. The HDD sizes were re-analyzed and changed during the construction phase due to coating application limitations caused by the original pipe size. Various alternatives were analyzed for coating application alternatives. The City, the Engineer and the Contractor were able to work together to secure a good solution for all parties. At present, the construction of both of these HDDs has been completed on schedule with minimal challenges. The detailed design process, construction process, and lessons learned will be discussed in this paper.

Section 1: Mary Rhodes Phase 2 Project Background

As Texas experiences some of the worst droughts in its history, the City of Corpus Christi (City) has worked hard to diversify their water sources to keep up with the needs of their residential and industrial customers. Implementation of the Garwood Water Right is the City's latest water supply strategy. The City purchased 35,000 acre-feet per year of the Garwood Water Right, which can be transferred from the Colorado River in 1998.

In 2002, the City contracted Freese and Nichols (FNI) to evaluate options for transporting the Garwood Water Right to the existing Mary Rhodes Phase 1 Pipeline. The existing Mary Rhodes Phase 1 pipeline and associated pump stations transfer water from Lake Texana in Jackson County to the O.N. Stevens Water Treatment Plant in Corpus Christi and were designed to allow upgrades for greater flow capacity. See Figure 1 below depicting the existing Mary Rhodes Phase 1 system as well as the Phase 2 project.

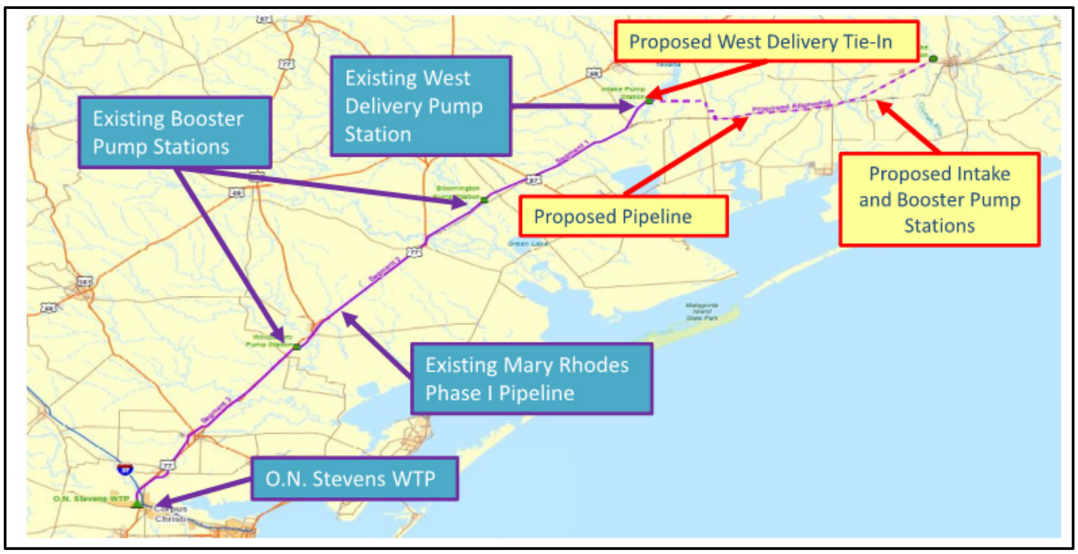

Figure 1: Mary Rhodes Phase 1 and 2 Systems

The Mary Rhodes Pipeline Phase 2 project includes a variety of components working together to obtain, settle, transport, and store the water contained in the City's portion of the Garwood Water Right. The project begins with a River Pump Station located on the west bank of the Colorado River in Bay City, Texas. The 46 MGD pump station will divert the water right into a 59 acre-feet, two-celled, HDPE and soil-cement lined sedimentation basin, as water quality studies showed the need to remove sediment prior to pumping to protect the pumps and pipe and to reduce energy use. See Figure 2 below for a schematic of the pump stations and sedimentation basins located on the Colorado River site.

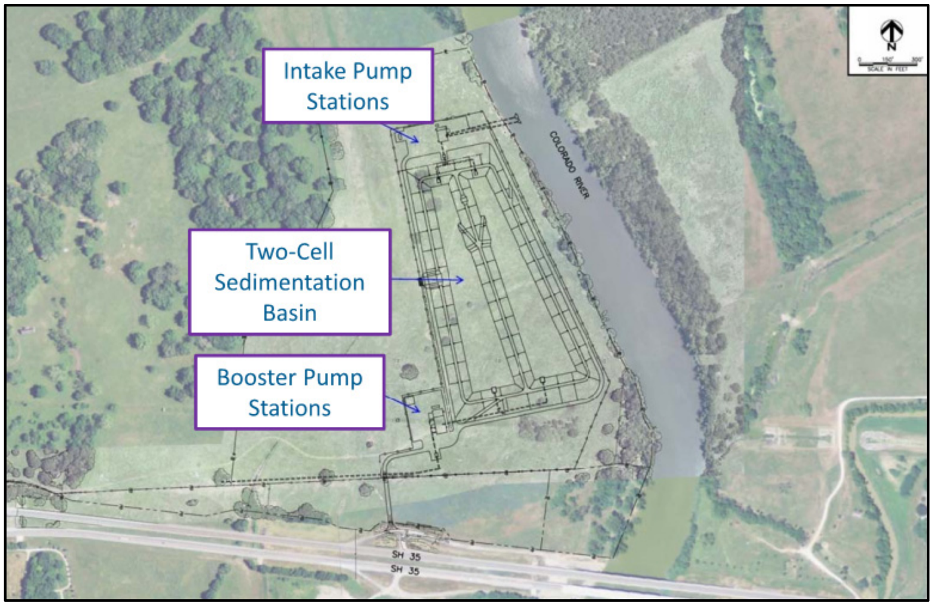

Figure 2: Mary Rhodes Phase 2 Pump Stations and Sedimentation Basin

After the sedimentation basin, the water is picked up by the 46 MGD Booster Pump Station to be pushed the approximately 42 miles through 54-inch pipe. In an effort to control costs, the City chose to bid both concrete cylinder pipe and steel pipe material alternatives. While the costs for both pipe materials were comparable, concrete cylinder pipe was the less expensive option. The City chose a route that parallels existing major roadways to help with ease of access and maintenance for the life of the project, as well as to control land acquisition costs. The 54-inch pipe discharges into a 6 MG balancing ground storage tank (GST). The Garwood water will travel by gravity from the GST to the tie-in with the existing intake header for the Lake Texana pump station into the Mary Rhodes Phase 1 pipeline.

The existing Mary Rhodes Phase 1 system includes two booster pump stations along the 101 miles of 64-inch pipeline. These booster pump stations have existing pumps and balancing reservoirs, however, it was originally envisioned that larger tanks and additional pumps would be added in order to provide the additional Garwood water, and open slots were left for this purpose. These improvements will be part of a later improvements project for the City.

The City utilized Texas Water Development Board (TWDB) funds for the design of the project, which provided a low interest loan for all planning and design tasks. Prior to release of these funds, the City had to secure all environmental and archeological clearance from the U.S. Army Corps of Engineers (USACE) and the Texas Historical Commission. One such permit acquired was a Regional General Permit (RGP) which requires trenchless installation under navigable waterways.

Section 2: Design of Two Large Diameter Horizontal Directional Drills

In order to comply with the Regional General Permit issued by the USACE for the project, the design team was required to cross two navigable rivers by trenchless methods. These rivers are the Tres Palacios River in Matagorda County and the Navidad River in Jackson County. The Tres Palacios River is approximately 100 feet in width and 40 feet deep and the Navidad River is approximately 300 feet in width and 25 feet deep. Three different trenchless options for installing the 54-inch pipeline were identified and investigated for these two crossings. These included traditional Auger Boring with a casing pipe, micro-tunneling, and Horizontal Directional Drilling (HDD). Auger Boring and HDD exhibits are shown in Figures 3 and 4. Auger Boring typically is a two-pass approach and has limited steering capabilities. When working within wet soils and below the ground water table, Auger Boring is typically not considered and is not generally used for installations requiring high accuracy. Auger boring was not investigated in detail for these specific trenchless crossings. Micro-tunneling can be accomplished with more accuracy but for the length of crossings in this project, this option is cost prohibitive as it requires very specific equipment that is costly to both mobilize and operate. With HDD installation, the pipe is typically not installed within a casing pipe, but pulled into a reamed hole that is drilled first using a guided pilot drill. Bore pits are not required for HDDs. Based on the analysis of multiple trenchless methods and the high probability for ground water adjacent to the rivers, HDD was selected as the preferred alternative for the two river crossings on the project.

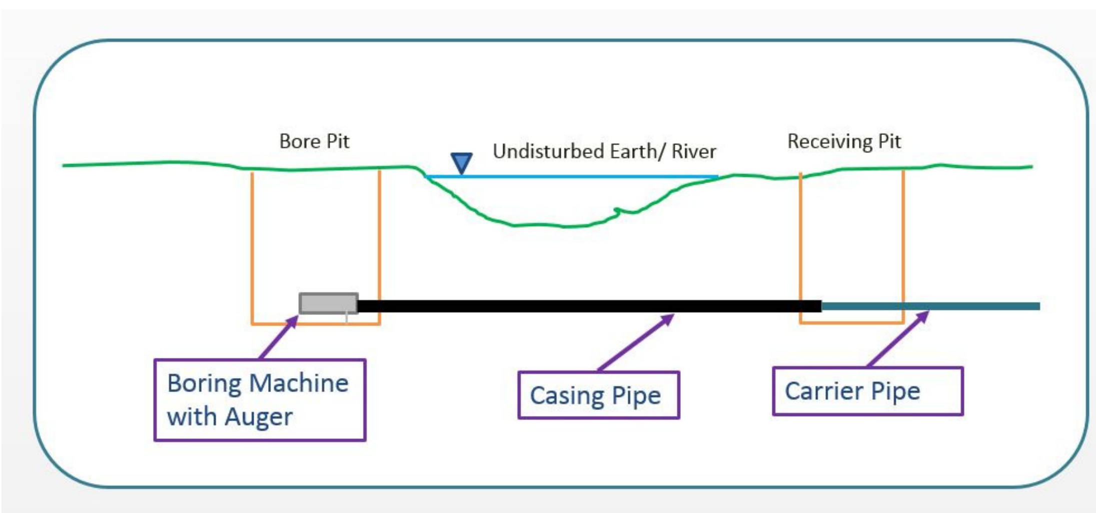

Figure 3: Auger Boring

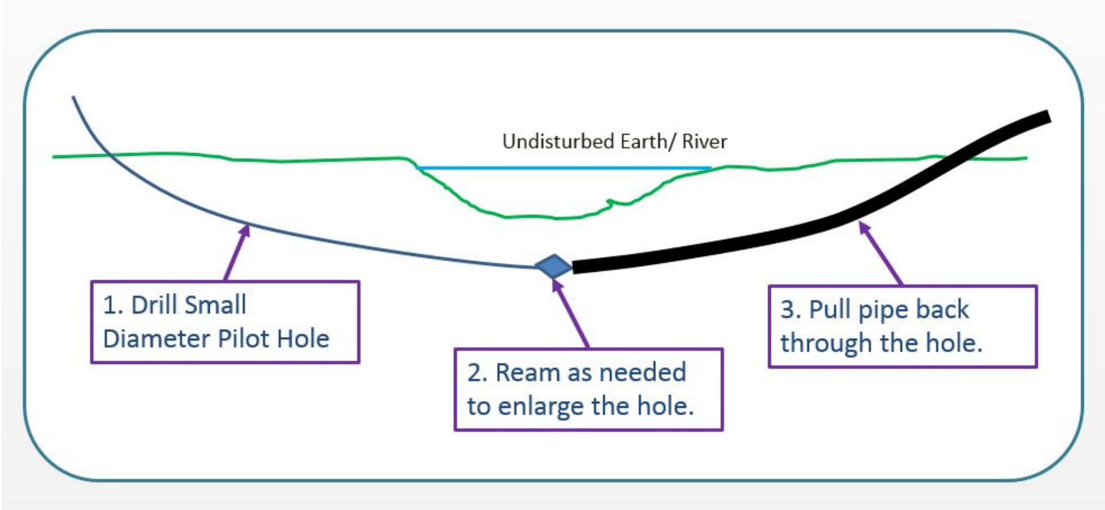

Figure 4: HDD

The size of the overall pipeline for the entire project is 54-inch diameter, which is on the high end of size ranges for an HDD installation. FNI coordinated with multiple HDD experts and contractors to review the constructability of the design and determine what pipe sizes were applicable for the HDD installations. The critical aspects of the HDD design include: entry and exit angles, minimum radius of the pipe, soil conditions, minimum depth and pipeline size.

The project team determined that a 200 PSI Pressure Rated 48-inch steel pipe should be used for the HDD crossings. Using 48-inch diameter pipe instead of 54-inch diameter pipe will allow a greater scouring velocity and reduce the need for line maintenance in the future. The decrease in pipe size very minimally affected the hydraulics of the pipeline. Originally, both steel and HDPE pipe materials were analyzed as options for the HDD. Due to the needed internal pressure rating required for the pipeline, the HDPE pipe would need to have a low dimension ratio and be extremely thick. HDPE pipe of this thickness was not commercially available or manufactured in the US. The HDD crossings are longer with a steel pipe than with an HDPE pipe because steel is less flexible and therefore has a larger minimum radius, but steel is the stronger-walled pipe which allows for a lower risk of buckling and/or deflection after installation.

The entry angle for the deeper Tres Palacios River was set at minimum 10 degrees and maximum 11 degrees. The entry angle for the shallower Navidad River was set at minimum 9 degrees and maximum 10 degrees. The exit angle for the Tres Palacios River was set at minimum 6 degrees and maximum 8 degrees while the exit angle for the Tres Palacios River was set at minimum 4 degrees and maximum 6 degrees. These angles were determined using a minimum radius of 100 multiplied by the diameter of the 48-inch steel pipe (4800 feet).

For typical open trench design, 25-foot deep geotechnical bores were drilled every half mile for all 42 miles of the project. To gather further soil information required for the HDD, additional 100-foot deep geotechnical borings were drilled on both sides of each crossing, as close as possible to the rivers and the proposed alignment. HDD installations can be both more difficult and more expensive when rock is present in the soils, and luckily, the additional geotechnical borings did not show any rock in the area. The soils in the area of both HDDs were typically varying types of sand and clay.

Another critical component of the HDD pipe design was the coating system. Once steel pipe was selected, multiple options of both coatings and linings were analyzed to select a resilient coating that would require limited maintenance in the future. Ultimately, a fusion bonded epoxy with an abrasion resistant overlay was selected for pipe coating. This coating system is robust enough to handle the maneuvering of the pipe during the HDD process and a field repair kit can be used to touch up any damaged areas.

Section 5: HDD Construction Challenges

Large diameter HDD projects are always a challenge and this was no exception. The construction of both HDD crossings required thorough planning and execution with appropriately sized equipment and experienced personnel. Managing the spoils disposal to keep up with the reaming operations was especially challenging since the volumes are enormous. Also the contractor had to take great care not to over-ream the hole (make too many passes), so as not to cause mis-alignments in the softer portions of the hole. Overcoming pipe buoyancy was also particularly important.

During construction, the shop coating applicator had challenges applying the specified fusion bonded epoxy coating. The machine which coats the pipe could not handle any pipe with a weight greater than 350 pounds per linear feet. The 48-inch 200 PSI steel pipe has a weight of 379 pounds per linear foot, making it too heavy for the coating machine. To rectify the issue, FNI looked at resolutions of other coating options, decreased pressure rating of the pipe, and decreased pipe size. The final option of 44-inch 200 PSI pipe was chosen to keep the strength and pressure class of the pipe the same, but to decrease the overall weight. With the decreased pipe size, velocity is still below 8 feet per second at the maximum flowrate, there is an increased scouring capacity in the line, and the hydraulic impacts are minimal.

The installation of the HDD pipe under both the Tres Palacios River and the Navidad River was accomplished with minimal incidents. The steel pipe was strung out along the pipeline easement and field welded prior to pullback. The easement acquisition team secured 2000 linear feet of lay down area for both HDD locations so all pipe could be welded in place and pulled continuously. The welding of the pipe took approximately 12 days for each crossing. The joints were coated externally and internally with fusion bonded epoxy, with an additional 30 mil abrasion resistant overlay at the external joints. The coating system was a 3M scotchkote system, #6233 with a #6352 abrasion resistant overlay. The lining system was a 3M scotchkote system, #124.The pilot hole was completed in 14 hours for each crossing. For the pullback, the pipe was held up by cranes and utilized foam rollers to keep from scratching the coating as shown in Figure 5. A primary Magnetic Guidance System (MGS) with a secondary Para-Track System was used for the tracking system.

Figure 5: HDD Installation

The Contractor and quality control inspectors examined the exterior of all pipe prior to pulling it through the reamed hole. In areas where coating had been scratched, abrasion resistant overlay was on hand to touch up where needed. This pulling through of the pipe was stalled to allow this overlay to dry. The pipe installed at the Navidad River required a large repair due to the foam on the roller support breaking as shown in Figure 6. This was repaired on site with a large quantity of abrasion resistant overlay before completing the HDD.

Each of the full HDD crossings were pulled within a 13-hour period. No rock was encountered during the construction and the soils, classified as sands and clays, remained consistent and ideal for HDD installation.

Figure 6: Broken Roller Support

Section 6: Project Path Forward

The pipeline portion of the project is currently 80% complete. The construction is being performed by a prime contractor managing the whole pipeline length with subcontractors for three individual, equal length sections. This contractor is also responsible for the construction of the tie-in to the 64-inch Mary Rhodes pipeline. A second prime contractor is responsible for construction of the two pump stations and the sedimentation basin. The two HDD's were completed by a subcontractor to the prime contractor with minimal delays. Each drill pullback was completed within 13 hours. Additionally, an experienced resident representative team is performing construction inspection on the project to help deliver a quality project for the City of Corpus Christi for years to come.

Lessons Learned from Horizontal Directional Drilling Installation of HDPE Sewer Forcemains in Anne Arundel County, Maryland

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Abstract

Anne Arundel County is currently in the process of replacing approximately 25,000 linear feet (LF) of sewer forcemains (FM) using horizontal directional drilling (HDD) for trenchless installation. The County has initiated a small diameter sewer FM replacement project to rehabilitate and/or replace small diameter sewer FMs that have reached the end of their useful life and/or have a history of breaks/failures. The County decided to use HDD to protect existing infrastructure and minimizing environmental disturbance and to streamline regulatory reviews and approvals. This paper focuses on six individual contracts in Anne Arundel County, Maryland that have utilized HDD. The challenges encountered on each of the contracts are highlighted and then summarized as lessons learned for general HDD installation. The conclusions from the lessons learned are summarized below:

- The more information that is gathered in the design phase the smoother the construction phase will be.
- The clearer the contract documents are the less room for change orders during construction**.**
- A good public outreach program is essential**.**

INTRODUCTION

Anne Arundel County (County) is located on the western side of the Chesapeake Bay, south of the City of Baltimore, Maryland. It has approximately five hundred and thirty-seven thousand (537,000) residents and over four hundred and fifteen (415) square miles of land. In terms of population, Anne Arundel County is fifth largest county in the state of Maryland. The majority of the population lives in the northern portion of the County as well as along the coast of the Chesapeake Bay. The elevations across the County range from mean sea level to approximately 300 feet above sea level. The topography of the County is broken up by various rivers and streams draining to the Chesapeake Bay. Due to the topography, it is difficult to have long gravity lines that convey sewage to the Water Reclamation Facilities (WRF); therefore, there are a large number of sewer pump stations (SPS) and sewer forcemains (FM). Currently the County maintains over two hundred and fifty (250) sewer pump stations and over fourteen hundred (1,400) miles of sewer forcemains that convey sewage to eight (8) WRFs. The existing sewer FMs have diameters ranging from 2-inch to 54-inches and the pipe materials include but are not limited to: cast iron, steel, ductile iron, polyvinyl chloride (PVC), high-density polyethylene (HDPE) and pre-stressed concrete cylinder pipe. The majority of the FMs are made of cast iron and were installed in the 1960s and 1970s.

BACKGROUND

In 1986, the County initiated the Sewer Main Replacement/Recondition Project (SMR/R), as part of the Capital Improvements Program (CIP), to ensure the adequacy of the County's wastewater collection and conveyance systems. The CIP program is controlled by the County's Department of Public Works and addresses the major repairs and the improvements of the FMs. Heery International, Inc (Heery) was hired by Anne Arundel County in 2006 to augment the County's Capital Improvement Program Management capability. Heery's project management team is currently assigned to the majority of the SMR/R's contracts. This paper covers the portion of the SMR/R Project that replaces the small diameter (under fifteen 15-inches) sewer FMs either due to breakage or based upon inspection results.

Since the year 2000, the County has replaced over forty (40) miles of existing cast iron, steel, and concrete pipe with HDPE pipe using HDD methods. Each of the FM replacement contracts are named for the sewer pump station that they originate from and all of the FMs terminate at a gravity interceptor manhole or a larger FM that conveys the sewage to the WRFs.

DECISION TO USE HDD TRENCHLESS TECHNOLOGIES

The majority of the FMs that are being addressed are located in County ROWs and/or utility easements. There are other utilities located in the ROWs and easements such as water, storm drains, gravity sewer, gas, cable, communications, and electrical lines. Most of the existing SPSs are located in residential neighborhoods and the sewer FMs are located in the residential streets. Open trench installation would require numerous street closures and/or detours that would negatively impact the residents. Open trench installation also results in more total disturbed area for a project and in the state of Maryland, disturbance of areas above certain thresholds have additional permitting requirements. Additional permitting can result in a longer design phase and delay the start-up of construction. Further permitting needs are also driven by many of the neighborhoods being located in the critical area (within a thousand (1,000) feet of the Chesapeake Bay).

Due to the County's desire to protect existing infrastructure, minimize environmental disturbance and to streamline regulatory reviews, the County has chosen to use HDD methods where feasible to replace small diameter sewer FMs.

LESSONS LEARNED ON SPECIFIC CONTARCTS

LESSON 1: PHYSICAL LOCATION OF EXISTING UTILITIES

The Manhattan Beach FM Replacement contract included replacement of approximately 5,000 LF of existing 12-inch cast iron pipe with 12-inch HDPE pipe from Manhattan Beach 1 SPS to an existing 36-inch sewer FM. The existing FM had severe external corrosion that caused several leaks within a short period of time necessitating replacement. The construction for this contract began April 2012 and ended in December of 2012. After the contractor had drilled the initial pilot hole, it was determined that the existing FM had not been marked correctly causing the pilot hole to be too close to the existing FM and valve vaults. Per County Standards, the new FM was required to be a minimum of 5 feet (horizontal and vertical) away from the existing FM and structures. The pilot hole location resulted in only 1.5 feet of separation. The contractor was required to relocate the drill rig and re-drill a new pilot hole in an alternate location. While the contract drawings did include a note indicating that the contractor was responsible for verifying all existing utilities, they were not specific about which utilities had to be field verified or at what distance away from the new FM the existing utilities had to be field verified. Additionally, the contractor damaged several sewer laterals while drilling the pilot holes for the new FM. The contractor did not test pit for all utilities that the new FM crossed.

In order to avoid the above referenced situations on future SMR/R contracts, all contract documents now clearly state that the contractor be responsible for test pitting any existing utility that the proposed FM crosses and within a specified distance away from the proposed FM. The County utilizes a utility location service (Dig Safe/Miss Utility) to mark all of the existing utilities along the proposed alignment before excavation can begin. The full time onsite third party inspector walks the alignment before construction mobilization to locate any existing utilities that may not have been shown on the contract drawings or missed by the utility locator. County contingent bid items have also been increased for additional test pitting for utilities that are not shown on the drawings. This allows the County, third party inspector and/or the design Architect/Engineer (A/E) to instruct the contractor to test pit additional locations where the new FM may be close to existing utilities or structures that are found in the field. Once all of the test pitting has been completed the A/E and County will review the proposed horizontal and vertical alignment with the contractor to determine if changes are required to avoid potential conflicts. By taking these measures the County has avoided damaging exiting utilities on contracts that came after this one.

LESSON 2: COORDINATION WITH OTHER CONTRACTS

The Cape Arthur V FM Replacement contract included replacement of approximately 1,770 LF of existing 10-inch cast iron pipe with 10-inch HDPE pipe from Cape Arthur V SPS to an existing gravity line manhole by HDD trenchless methods. The existing FM had experienced several leaks due to severe external corrosion that necessitated replacement. The construction for this contract began in July 2013 and was completed March 2014. The County was in the process of upgrading the SPS while the new FM replacement contract was in design. The County gave the contractor doing the SPS upgrades a field directive to install a wye with an isolation valve on the existing FM so that the new FM replacement work could connect directly to the isolation valve and not require a pump station shut-down. This saved the County time and money. It also mitigated the risk of future shutdowns because an SPS shutdown was not required to tie into the existing FM. Additionally, during construction the onsite inspector located a storm drain crossing the road that intersected the new FM, which was not shown on the contract drawings. The storm drain was at the same elevation of the proposed FM requiring a change in the proposed FM's profile. This change created a high and low spot in the new FM. Because of this, a new Air Release Valve (ARV) and vault had to be installed at the high spot at the County's expense. This challenge is another example of Lessons Learned No. 1 – Physical Location of Existing Utilities.

The County coordinates its current Upgrade/Retrofit SPS and FM Replacement contracts to make the work more efficient. If another contract is in design or construction at the SPS where the FM replacement contract is going to take place, provisions are made so that the new FM can be connected to the existing FM without having to shut down the SPS.

LESSON 3: REQUIRE SUB-SURFACE INFORMATION

The Belvedere Yacht Club FM Replacement contract included replacement of approximately 3,100 LF of existing 6-inch ductile iron pipe with 6-inch HDPE pipe from Belvedere Yacht Club SPS and tied into an existing larger FM by HDD. The existing FM was experiencing severe external corrosion necessitating replacement. The construction for the contract began in September 2013 and was completed in May 2015. The existing FM was located in a neighborhood near the water (Magothy River) and the soil is mostly sand with a shallow water table. In one of the deeper excavations for the new FM, the contractor could not maintain a stable excavation to install a blow-off valve vault. The excavation required sheeting and shoring because it was located near a neighborhood road and there was not enough space to allow a benched or sloped excavation. A geotechnical data report with boring logs was provided to the contractor with a soil boring at the location of the proposed blow-off valve vault. Eventually the contractor had to install the new FM along a different alignment that did not require an excavation that deep (approximately 20 feet). During construction there was a conflict with an existing storm drain inlet that caused the alignment to have to be re-engineered by the design A/E. The storm drain was not shown on the contract drawings. This challenge is another example of Lessons Learned No. 1 – Physical Location of Existing Utilities.

The County now typically requires the Design A/E to provide a geotechnical report with the contract documents for all FM replacement contracts. The report includes several borings along the new FM alignment that lets the contractors know the soil type and water table level throughout the alignment. The report also gives recommendations for sheeting and shoring in the excavations.

LESSON 4: THINK ABOUT DOWNSTREAM STRUCTURES

The Valley Road FM Replacement contract included replacement of approximately 1,890 LF of existing 6-inch ductile iron pipe with 6-inch HDPE pipe from Valley Road SPS to an existing manhole (MH) by HDD. The existing FM was experiencing severe external corrosion necessitating replacement. The construction for this contract began in October 2013 and was completed in May 2015. During construction it was determined that the discharge MH where the new FM connected was structurally unsound. The discharge MH conveyed flow from three other sewer pump stations and could not easily be taken out of service. The existing discharge MH walls had been scoured away from the existing FM discharge. The contractor was able to construct a form in the MH to install a structurally stabilizing liner (PermaForm) without taking the MH out of service. The contractor also installed a 90 degree elbow on the discharge of the new FM so that the flow was directed downward and not towards the wall of the MH. During construction of the new FM an ARV vault and Blow-off valve vault were found on the existing FM. The contract documents did not include language about demolishing the existing vaults, so the County had to include that work in another contract. This challenge is another example of Lessons Learned No. 1 – Physical Location of Existing Utilities.

The County now requires that any manhole where new FMs discharge to be lined or rehabilitated such that the discharge end direct flow towards the floor of the manhole if it penetrates above the bench of the manhole. The discharge manholes typically have more turbulent flow characteristics that releases more hydrogen sulfide, which causes more corrosion than manholes with laminar flow. The County also requires the contractor to line the two downstream manholes from the discharge manhole. With each FM Replacement contract at least three manholes are being rehabilitated. This will reduce the number of manhole failures in the future for the County.

LESSON 5: HAVE PROCUREMENT METHODS IN PLACE IN CASE OF EMERGENCIES

The Twin Harbors IV FM Replacement contract included replacement of approximately 450 LF of existing 6-inch ductile iron pipe with 6-inch HDPE pipe from Twin Harbors IV SPS to an existing FM by HDD trenchless methods. The existing FM was experiencing severe external corrosion and several failures during the bid phase of the contract necessitated immediate replacement. The County had to expedite the purchase order for the contract in order for the contractor to mobilize early to the site. Within a couple weeks of the new break the contractor had installed a temporary bypass line to take the existing FM out of service while the submittals were in review.

If a sewer FM breaks while the contract is in design the County has procedures set in place for an emergency expedited procurement process.

LESSON 6: HAVE A GOOD COMMUNITY OUTREACH PROGRAM

The Whitehurst FM Replacement contract included replacement of approximately 3,425 LF of existing 6-inch ductile iron pipe with 6-inch HDPE pipe from Whitehurst SPS to an existing MH by HDD trenchless methods. The existing FM was experiencing severe external corrosion necessitating replacement. The construction for this contract started in July 2014 and was completed in May 2015. During construction there were several challenges that had to be addressed such as changing the vertical profile, coordinating with the homeowner's association and the board of education, breaking existing sewer laterals and gravity mains. The subcontracting driller reached an intersection where the new FM was supposed to go underneath all of the existing utilities in the road (water line, gas line, storm drain, gravity sewer line, and existing sewer FM). After the contractor located all of the existing utilities they determined that they had enough clearance to meet county requirements and go above the deeper utilities and below the shallower utilities. The contractor submitted a Request for Information (RFI) with a new vertical alignment. A new blow-off valve had to be added to the alignment and the location of the ARV was relocated to the new high spot. In order to install the new ARV vault, the contractor needed to disturb a portion of resident's yard with landscaping that was in the County's Right-of-Way. The County proactively engaged in conversation with the resident and worked out a plan that was agreeable to both parties. The contractor was able to trim some trees that resident needed trimming, which in turn allowed for better access to set the ARV vault with a crane. The contractor restored the property back to its original condition. This was done at no cost to the County because of the cost savings from not having to dig out the deeper excavations.

While drilling a pilot hole on another section of the new FM the contractor broke the existing gravity sewer line that was running parallel to the new force main. The markings showed the existing gravity line to be further away incorrectly. The onsite inspectors periodically checked the gravity line manholes in the sections where the drilling is taking place, so this break was found quickly and repaired the same day. The break occurred right before a holiday weekend, so if the inspector had not checked the manholes this could have ended up being a large spill on a holiday weekend. The County coordinated with the residents that were affected by the break to reduce flow in the gravity line for approximately two hours while the work was being performed.

For the last section of the new FM, the contractor had to curve around an existing manhole in the road and then go off of the road into a twenty foot utility easement located in the back of five different properties ending at the sewer pump station. There were decorative landscape koi ponds in one of the backyards that the new FM passed directly underneath them. The contractor dug out intentional relief pits so the no fracking would occur in the resident's yards or damage the koi ponds.

The County coordinated with the Board of Education for the contractor to be on school grounds because the discharge manhole was located in an elementary school playground. The contractor performed the work in the school yard when recesses where not in session or at times when the school was out. The County also coordinated with the homeowners association on the paving repairs in the neighborhood.

The County proactively informs residents and business owners of construction projects with flyers, site visits, and public meetings. When a project is going to directly impact a specific property, the County engages in conversation with the property owner to inform them of what will be happening, sets expectations and provides contact information. This has helped many projects be completed in a timely manner.

SUMMARY OF LESSONS LEARNED

The lessons learned have resulted in measures that the County now takes to reduce the risk of the challenges faced during design and construction.

Lesson 1 – Physical Location of Existing Utilities. The County now clearly states in all of the SMR/R contract documents that the contractor be responsible for test pitting any existing utility intersecting or parallel to the proposed FM. The utility locator marks the exiting utilities in the field before any excavation or drilling can begin. The onsite inspector walks the alignment before construction mobilization to locate any existing utilities that may not have been shown on the contract drawings or missed by the utility locator. County contingent bid items are increased for additional test pitting if any existing utilities are found in the field. This allows the County, inspector and/or A/E to instruct the contractor to test pit additional locations where the new FM may be close to existing utilities and/or structures that are found in the field.

Lesson 2 – Coordination with Other Contracts. If another contract is in design or construction at the SPS where the a new FM replacement contract is going to occur, provisions are made so that the new FM can be connected to the existing FM without having to shut down the SPS multiple times.

Lesson 3 – Require Sub-surface Information. The County now typically requires the A/E to provide a geotechnical report with the contract documents for all FM replacement contracts. The report includes several borings along the new FM alignment that lets the contractors know the soil type and water table level throughout the alignment. The report also gives recommendations for sheeting and shoring in excavations.

Lesson 4 – Think about Downstream Structures. The County now requires that any MH where new FMs discharge, be rehabilitated and that the discharge end direct flow towards the floor of the MH if it penetrates above the bench. MHs where FMs discharge at typically have more turbulent flow characteristics. This turbulent flow releases more hydrogen sulfide, which in turn causes more corrosion than MHs with laminar flow. The County now also requires the contractor to line the two downstream MHs from the discharge MH.

Lesson 5 – Have Procurement Methods in Place In Case of Emergencies. If a sewer FM breaks while the contract is in design the County has procedures set in place for an emergency expedited procurement process.

Lesson 6 – Have A Good Community Outreach Program. The County proactively lets residents know what construction projects are going on in their neighborhoods with flyers, site visits, and public meetings. When a project is going to directly impact a specific property the County engages in conversation with the property owner to discuss what will be happening, what to expect to see, how long it will last, and contact information to ask questions. This has helped many projects be completed in a timely manner and with a reduction in complaints from the community.

CONCLUSIONS

Over that past fourteen (14) years that the County has been utilizing HDD methods to install sewer FMs it has carried over lessons learned from the previous contracts. Below is a general summary of the lessons learned that will help other municipalities or utility owners that are considering using HDD trenchless methods for pipe installation.

The more information that is gathered in the design phase, the smoother the construction phase will be. Although there are certain unforeseen issues that will arise on any project, the more information that can be gathered during design, generally, the construction will likely have fewer problems that will result in schedule and cost savings. Several key ways to gather information are listed below.

- Require a complete survey along the entire alignment of the new FM. This will pick up most of the utilities.
- Walk the alignment several times to identify any utilities that may have been missed. Talking with residents that have lived in the neighborhood along time can be helpful to find utilities that are not easily found.
- Require soil borings at each location where excavations will occur and in between excavations if they are more than 500 feet apart. This lets the contractor know the soil types and water table information.
- Have the utility easements staked out before construction begins so the contractor can see what if any obstructions may be encountered.
- Have the design A/E test pit all utility crossings or have language in the contract documents that state that the contractor is responsible for test pitting every utility that the new line will cross and any existing utilities or structures that are within a certain distance of the new line, generally within 5 feet.

The clearer the contract documents are the less room for change orders during construction. Allow room in the contract documents to deal with unforeseen issues that arise in construction and clearly spell out who is responsible for addressing those issues. Several key things that the County now includes in their contract documents are listed below:

- Have contingent line items in the bid for items that commonly are needed during construction such as test pitting, additional excavation, backfilling, compaction testing, sub-base, paving, curb and sidewalks.
- In the measurement and payment sections, be clear on how the contractor can be paid for Work such as if they can bill for stored materials or only once it is installed or if a percentage can be paid for materials that are onsite. Clarify if it is only the amount of pipe that is installed in the ground that will be paid for or if it is by them amount of pipe that was delivered to the site.
- Clearly show on the drawings all utility crossings. Call out what the contractor is responsible to locate in the field and who they have to coordinate with to get underground utilities marked.
- Show the plan and profile at a scale that can be read easily, no more than 1"= 40'.

A good public outreach program is essential. The more proactive the public outreach is to let people know what is going on and who they can contact to get questions answered, the less likely there will be issues with the residents. Several key things that the County tries to do to let the residents know what is going are listed below:

- Hand out flyers or door hangers to all of the residents where the work will be taking place. The flyers describe the nature of the work, why it is being done, the positive benefits for the residents and who they can contact with questions.
- The County meets directly with residents where the work is on or very close to their property so that they know that the County is reaching out to them. If they are dissatisfied with what is going on and the onsite personnel cannot answer their questions the County has a Public Relations group they can get in contact with and they will try to resolve any legitimate claims.
- For larger contracts affecting an entire neighborhood a public meeting is used to inform the residents of what is going and they can give their input at that time.
- The County also coordinates with the Board of Education for contracts that will affect bus routs and stops. The County will schedule/coordinate the work to minimize any impacts to the bus routes. Specially, if a contract is located on a bus route the working hours are reduced to times when the buses are not in service.

HDD River Crossing Improves Reliability of Water System and Meets Growing Demand

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Abstract

This paper discusses a 30" HDPE new water main installation project installed by the horizontal directional drilling process under a river. It will discuss the challenges of the installation including a steep angle of entry, significant elevation variances from one side to the other, varying soil conditions along the path of the bore, environmental challenges and more. Additionally, the new line was installed in a corner where 3 municipalities met and benefits related to good communications between all involved will be discussed. It will prove how the right material, the right equipment, and partnership between all with a staked interest in projects can overcome challenges and produce a quality product for the end users. It will also point out how bore fluid choice, hydraulic pressures while boring and soil condition as well as terrain assessment prior to beginning the project are instrumental to a successful project of this nature. Also, dealing with 3 municipalities, 2 state agencies and a developer of an assisted living facility within the confines of the jobsite brought additional administrative and physical challenges that will be discussed. Additionally, the owner's needs and desires including, but not limited to, time constraints, environmental concerns, public relation concerns, and the ultimate need for this line as a substantial and important feed for a growing territory will be discussed. Ultimately, this paper will clearly exhibit that planning, cooperation, communication and executing to plan can lead to a very successful project even though many challenges are encountered along the way.

INTRODUCTION

Citizens Energy Group (CEG) provides safe, high quality water service to approximately 400,000 homes and businesses in the eight county Indianapolis area (Marion, Johnson, Morgan, Hendricks, Boone, Hamilton, Hancock and Shelby).

Water is transported via 4,000 miles of pipeline from nine water treatment plants strategically located near primary water sources including the White River and

Geist, Morse and Eagle Creek Reservoirs. The Indiana Central Canal is also an important water source for the city. (CEG 2015)

Hamilton County is one of the fastest-growing counties in the United States. According to the U.S. Census, the county's population increased by 8.1% from 2010 to 2013. (US Census 2013) In order to meet the growing demand of this growth, It was necessary for CEG to install additional transmission main line to complete a 'loop' within their system providing additional feed for even more growth. This was accomplished in three phases. Phase 2 involves crossing the White River and is shown in figure 1.

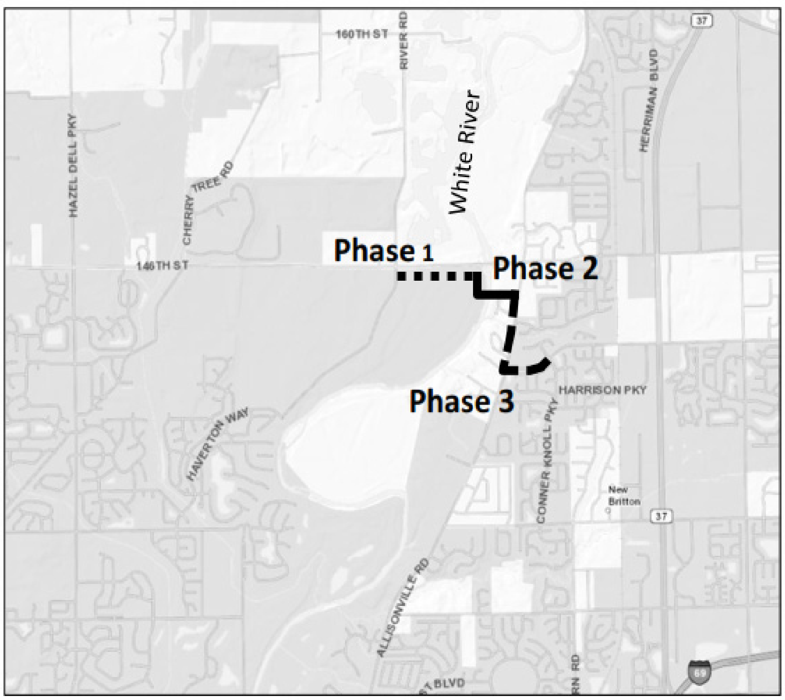

Figure 1. Project Location

MATERIAL

To complete the loop in their system required crossing the White River with a transmission pipeline of adequate size, strength and flexibility. 30" HDPE was chosen to meet these vigorous requirements. 1,100 feet of 30-inch diameter, solid wall, PE 4710 high-density polyethylene (HDPE) pipe with a dimension ratio (DR) of 13.5 was specified for this project. McElroy fusion units were used to heat-fuse sections of HDPE pipe to construct a leak-free line. High density polyethylene (HDPE) has been used for municipal and industrial water applications for almost 50 years. PE 4710 pipe has the properties that make it an extraordinary choice for complex, trenchless projects such as this. It is flexible, highly ductile, durable and has leak-free, fully-restrained fused joints. HDPE pipe's flexibility and exceptional bending radius allows it to adapt to the challenging site conditions. For this DR 13.5 pipe project, the minimum bending radius was 25 times the outside diameter. (PPI 2015) Typical pressure for this Citizens Water transmission line is 115 psi at a rate of two to four million gallons a day.

DRILLING EQUIPMENT

A Vermeer 330x500 boring machine utilizing a ParaTrack® 2 guidance system was chosen for this project. Because, the D330x500 is within legal dimensions in transport mode, it meets the needs of the industry for increased flexibility and ease of transport any time of day. The unit also offers a quick setup time because all components, including the engine, hydraulics, operator's cab, crane, and breakout system, are part of the machine. With 50,000 ft-lb of rotational torque for drilling in difficult ground formations and turning large back reamers, and 330,000 pounds of push and pullback force, the D330x500 packs an impressive amount of muscle for those long and large-diameter drilling projects. The machine also increases efficiency by allowing a single operator to run everything from the climate-controlled operator's station. All functions, including transport, drilling controls, crane, and drilling fluid controls, are operated from the cab of the D330x500. Drill pipe up to 32 feet (9.75m) in length can be utilized by the D330x500. (Vermeer 2015)

The ParaTrack ® Steering Tool system was utilized by INROCK, the bore steering subcontractor assigned to this project, to provide an accurate method of verification of the positioning of the drill head when magnetic interference or excessive depth is encountered as was the case with this crossing. Typical walkover locating would not be feasible for this bore due to the depth, the differing soil conditions and of course the body of water being crossed. HDD Guidance technology, developed and manufactured by Vector Magnetics, as utilized on this bore, is rapidly being adopted as the HDD industry's recognized standard for underground guidance accuracy. Lateral and vertical accuracy measures 0.2ft/10 ft. of bore hole depth (2%) (Inrock 2015)

Additionally, a Mud Technology MPCT 1000 'Mud Maxx', two Vermeer SA400 mud pumps along with high volume bentonite were instrumental to the success of this installation.

CONTRACTOR

Miller Pipeline participated in a RFP process through Citizens Energy Group was ultimately chosen as contractor on this project.

OBSTACLES

There were many obstacles from design, construction and administrative perspectives that were to be overcome for success of this project.

DESIGN

From the design perspective, accessing the path of the bore to obtain adequate geotechnical information was difficult at best. As seen in figure 2, the east side of the project was at an elevation considerably higher than the west side. It dropped off quickly from east to west (50' of drop within 100') before encountering wetland, the river, a levee and then park property.

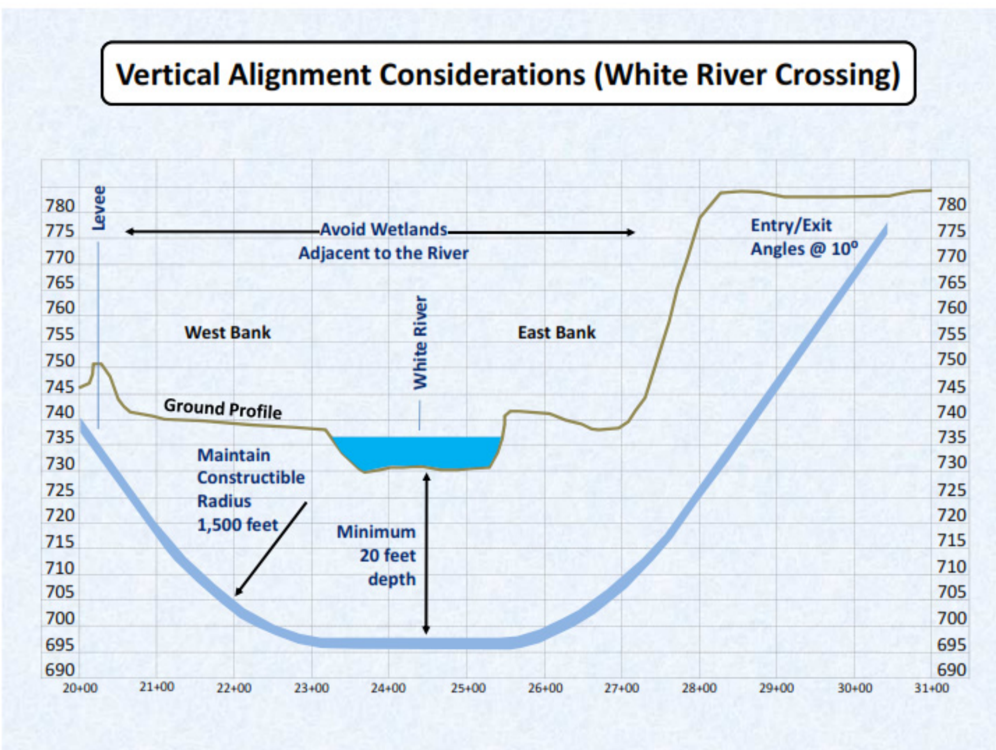

Figure 2. Elevation and other difficulties

Different methods were attempted to obtain important geotechnical information. Among these, were ground penetrating radar (GPR), seismic data collection, and conventional soil borings.

Ground penetrating radar utilized a 100 MHz bistatic antenna system. Due to the moist cohesive soils and shallow ground water which attenuated the energy of the gpr signal rapidly, quality of collected data was low and resulted in inconclusive results. Ground penetrating radar, while effective in sandy, silty, less cohesive soils, proved ineffective in this application. (DLZ 2013)

Seismic data collection provided data that was consistent with other evidence and experience encountered by contractor in geographical area—bedrock existed below bed of river. Again, while very useful when the data can be collected in substantive quantities, seismic data was not able to provide much new insight in this installation.

Additional data obtained by soil borings indicated varying soil conditions would be encountered in bore path. These included clay, sand, cherty limestone, boulders and silty soil—this wide array of soil conditions proves to be very challenging when utilizing HDD with large diameter such as this. Tooling used in the industry is very technically manufactured to be effective for specific conditions.

Clearly, these conditions made an already difficult project even more difficult to execute as strategic decisions, such as deciding to push ream each segment and what type of tooling to use were based on these findings and made to mitigate risks associated with these conditions.

The final step to the design was in reality the first step of the construction process. INROCK did a complete survey of the entire path to suggest a preferred path that should be followed. The end product was a design path that the owner, design engineer and contractor all agreed upon. This is a crucial step in the collaboration of this team, and with virtually all projects. There must be consensus that the end product firstly meets design criteria with respect to product strength and radius parameters, secondly, meets the needs of the owner, and lastly, and equally important can be constructed in such a manner.

Administrative

Several administrative hurdles would be faced with this project. The project was located in an area immediately bordered by 4 municipal authorities: City of Carmel, City of Fishers, Hamilton County and the City of Noblesville. As can be seen in figure 3 below, coordination with these four agencies coupled with the Indiana Department of Environmental Management and the Environmental Protection agency would be imperative to a successful project.

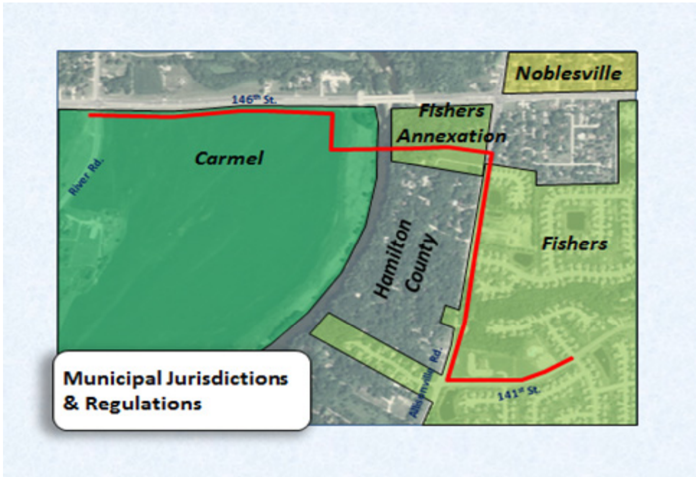

Figure 3. Multiple Jurisdictions

As would be expected, all 4 municipalities had their own unique rules and regulations related to excavation, water supply, road cleaning, erosion control, permitting and more. The coordination and cooperation by owner and contractor were vital. For the sake of this project, 60 years of partnering between Citizens Energy Group and Miller Pipeline lends itself well to the trust and commitment required.

CONSTRUCTION

While the design of the project remained a work in progress as differing conditions were encountered and the administrative challenges continued throughout, the construction phase had to carry on concurrent with all other aspects in order to meet the owner's schedule.

The construction team mobilized the aforementioned equipment in addition to other conventional equipment such as track excavators, vacuum trucks, and other miscellaneous equipment which took approximately 4-5 days to complete. They began the pilot bore from the west side of the river.

After 10 days, the pilot bore was complete and exited the ground at an acceptable angle and depth to provide a tie in that was at a reasonable depth for the contractor and the owner's future access.

The initial ream (24") was pushed from the west side to the east side and completed in 5 days. It was critical to all involved that these processes, in this phase and subsequent phases, were not rushed. As stated earlier, with the changing soil types, it was not possible to have tooling in the ground that was optimal for all conditions encountered. As such, the recipe for success in this case was patience.

The second ream (38") was pushed from the west side to the east side and it, too, was completed in 5 days.

The third ream (42”) was again pushed from the west side to the east side and due to the overall smaller cut was completed in 2 days.

The fourth and final ream (48”) was completed in 2 days. The hole was then swabbed to ensure that an adequate hole that would require minimal pull pressure existed. It is absolutely critical that great care and attention to detail is used throughout the entire process.

Finally, the boring machine was moved to the other side of the project to pull the pipe in. As indicated earlier, the reamers were pushed from the west side using the boring machine to turn the reamers while 2 excavators on the east side of the river pulled them toward the East. Figure 4 depicts the final alignment of the pipeline. The pipe was ultimately successfully hydrostatically tested and tie-ins were made.

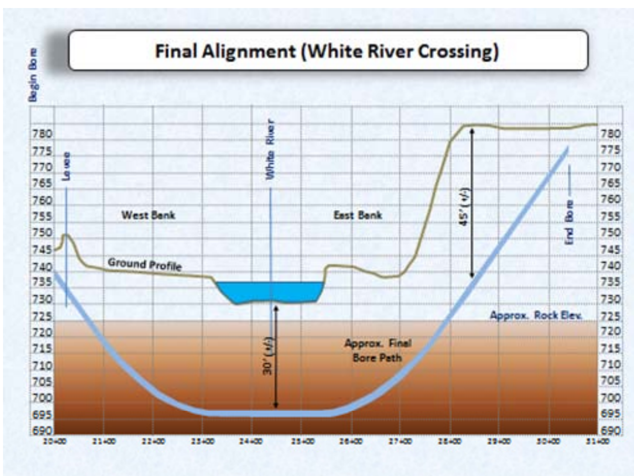

Figure 4. Final Alignment

CONCLUSION

In conclusion, It is apparent that a project such as this requires a partnership of all members who have a stake in it. In this case, The owner, design engineer, and contractor worked together to install a pipeline in such a way to minimize environmental issues, maintain good relationships and communications with municipalities and agencies involved while meeting budgetary and schedule expectations of the owner and it’s customer.

There were many examples of collaboration being the key to the success of this bore. Among these, consensus of the team related to design was critical. Equally important was the understanding of the owner and the engineer that patience must be exhibited to *help* insure success. Finally, daily communication among and across all parts of the team was absolutely essential. From discussing contingency plans, consistent flow of information to the Indiana Department of Environmental Management, and monitoring of water quality within the river, these efforts were every bit as important to the overall success as the construction itself.

This team approach utilizing design through construction was instrumental to the success of this and future projects of its kind.

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Thermal Contraction Lesson Results in Steel Tunnel Liner Damage

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Abstract

The Silver Lake Reservoir Complex located in Burbank, CA is one of several projects to replace open storage reservoirs in the Los Angeles area. This complex included replacement of the open reservoirs with enclosed reservoirs plus tunnels and open cut pipelines to accommodate the upgrade. One of the main components to the project included construction of a segmented concrete tunnel casing under Griffith Equestrian Park to the Silver Lake Complex and installing a 96” x .563” x 3,200’ butt welded steel tunnel liner which is the subject of this paper. The project was performed during the hot season (August) and mandated expedited/around the clock installation due to a delay of the preceding work. The design disallowed the use of welded pipe attachments for temporary support and blocking during installation which in turn dictated the use of block supports independent to the pipe. There were also no expansion joints or means to accommodate thermal contraction expected during the installation. Following the expedited pipe installation approximately 2,000’ of pipe was installed within the first 11 days and supported on nonorganic/plastic lumber intended for dead load support. During installation the pipe butt joints were stabbed into a steel backing ring and tack welded or root welded to provide a temporary connection. The project Specifications required SMAW welding (stick/manual welding) which limited the finish welding progress rate and therefore welding lagged considerably behind the installation. While in process and after installing about 2000’ the temporarily tacked/rooted joints began to pull apart as a result of thermal contraction. This paper explores the tunnel liner installation execution means and methods, the resulting thermal contraction causes and the suggested options to avoid future occurrences of this type to future tunnels.

INTRODUCTION

The Silver Lake Reservoir Complex is a $242 Million project located in Burbank, CA and is one of several projects to replace open storage reservoirs in the Los Angeles area. The US Environmental Protection Agency mandated that all open storage reservoirs used for drinking water be protected by covering or bypassing. Without protections, these reservoirs are exposed to surface runoff, birds, insects, animals, algae growth, and human caused contamination. A trunk line which includes both tunnel and open cut steel pipelines connects two Silver Lake reservoirs which have a combine capacity of 100 Million Gallons and creates the complex. The Silver Lake Reservoir Complex general location map is provided in Figure 1.

Figure 1 Silver Lake Reservoir Complex general location map

One of the main components to the project included construction of a 120” diameter segmented precast concrete tunnel casing under Griffith Equestrian Park to the Silver Lake Reservoir Complex and installing a 96” x .563” x 3,200’ butt welded steel tunnel liner through the tunnel and is the subject of this paper. The steel liner was located within the concrete casing providing a minimum 6” annular space which was later filled with 1000 PSI grout. After hydro testing both ends of the tunnel pipe were connected to the existing system by conventional cut and cover methods using bell and spigot welded steel pipe.

THE CONSTRUCTION TEAM

National Welding Corporation was responsible to assemble, fit and weld the tunnel liner as a subcontractor to Michels Corporation. Michels Corporation was the General Contractor of the overall tunnel project and self-performed most of the key project elements including tunnel excavation, annular grouting and oversight of all other activities. Ameron International prepared the steel tunnel pipe shop drawings, fabricated the pipe and provided shipping to site. Los Angeles Department of Water and Power is the project owner.

TUNNEL LINER PIPE DETAILS

The 96” welded steel liner had a .500” mortar lining and the exterior had a 1” mortar coating over an exterior tape wrap. Each finished pipe measured just over 2” thick, was 40 feet in length and weighed approximately 46,000 lbs. The coatings and linings were held back 9 inches at the pipe ends to facilitate the assembly fitting and welding of the pipe joints. The joint design was a butt weld with split steel backing to allow assembly and welding within a tunnel.

SPECIFIED PIPE BLOCKING DETAILS

The pipe blocking was specified to be a 6” pipe insulator commonly referred to as casing spacer/ isolators, Figure 2. However this type of spacer created concerns for use in this application including 1) fixed dimensions that cannot easily be changed to accommodate expected variations in tunnel elevation at different stations. 2) The spacers are attached to the pipe as permanent bands which requires a much smaller spacer diameter than the casing inside diameter in order have enough clearance for installation into a tunnel 3) these spacer would not block the pipe from flotation or movement during annular grouting 4) The method for installing this style of blocking is to jack/slide the banded pipe on the tunnel floor while adding each pipe length at the tunnel portal. This becomes an issue due to the tunnel length and the cathodic protection system which includes a full circumference expanded titanium mesh located every 40 feet (at each pipe joint) throughout the tunnel.

Figure 2 Casing Spacer/isolators

ALTERNATE WELDED STEEL BLOCKING

The team provided an alternative welded steel pipe support which was electrically isolated from the concrete casing by Ultra High Molecular Weight (UHMW) plastic pads and each joint could be custom fit to the required elevation, Figure 3. As this approach included intimate blocking of the pipe between the ceiling and floor of the tunnel this would alleviate the concern about the pipe floating during grouting. This option was not acceptable as having welded attachments to the steel liner was undesirable to the owner.

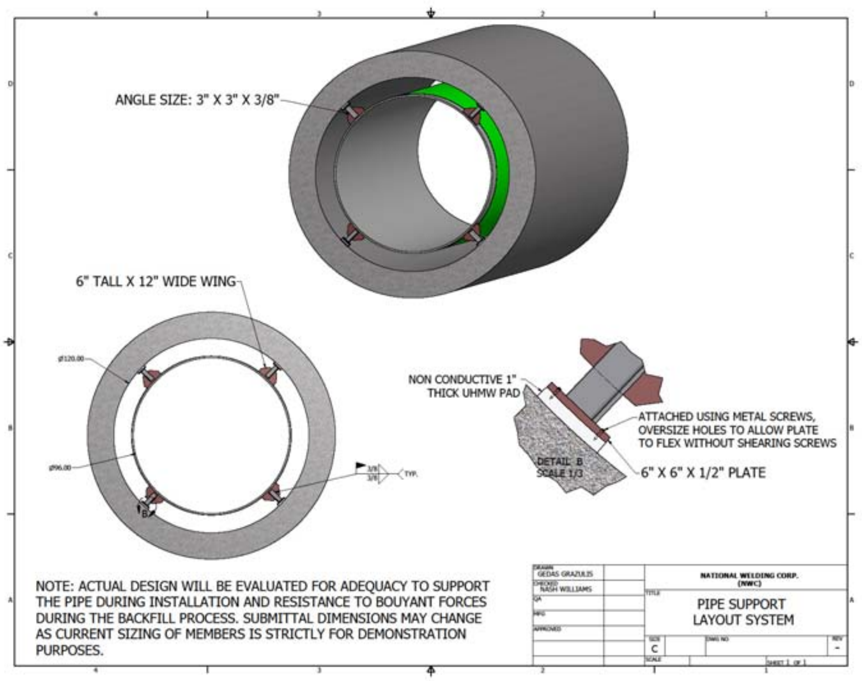

Figure 3 Alternate welded steel pipe support

ALTERNATE PROPOSED INDEPENDENT HDPE BLOCKING

The Team proposed a second alternate blocking method using independent adjustable HDPE (high density polyethylene) blocks set at the end of each pipe joint. This method was designed with a 4 times safety factor for dead load based on the 46,000 pound pipe sections, however it did not provide for axial movement. The cathodic protection mesh was located at every pipe joint including under each joint therefore it would have been damaged by axial movement anyway. This nonorganic, HDPE alternate achieved the isolation, non-welded attachment and adjustability required by the designer which was ultimately accepted and subsequently utilized on the project Figure 4a and b. Intermediate masonry bulk heads were also installed and provided some pipe support but were primarily intended to create separate cells for the grouting operation.

Figure 4a and b HDPE Adjustable Pipe Blocking System

TUNNEL PIPE WELDING PROCEDURE

The specified method of welding for this project was SMAW (Shielded Metal Arc Welding) commonly referred to as manual stick welding, Figure 5a. Each pipe seam included over 25 pounds of weld metal which was anticipated to cause substantial delays to the tunnel liner completion. Therefore FCAW-G (Flux Cored Arc Welding with Gas) commonly referred to as semi-automatic welding which is much faster, cleaner and has many superior properties to the originally specified process was submitted, Figure 5b. This proposed change to the Specifications was found to be an unacceptable deviation from the Specification. The SMAW welding was utilized and on the first 2000 feet the welding progress rate lagged several weeks behind the installation. The welding time for SMAW was measured throughout this project and found to be 2.5 to 3 times longer than FCAW welding.

Figure 5a and b SMAW (Stick Welding) and FCAW (Flux Cored Arc Welding)

PIPE STORAGE

The pipe had been fabricated ahead of time and subsequently stored in the California desert prior to shipment to site, Figure 6a. Due to the limited on site storage only a few day's supply of pipe was delivered to the site as the project proceeded, Figure 6b.

Figure 6a and b Pipe Manufactures Offsite Storage and Site Storage

EXECUTION OF THE LINER INSTALLATION

The project was performed during a hot season (August and September) and mandated expedited/around the clock installation due to delays of the preceding work.The pipe was lowered by crane to the installation carrier at the tunnel shaft then transported to the installation location beginning at the far end of the tunnel, Figures 7a, b and c plus Figures8a, b and c.

Figure 7a, b and cPipe Installation

Figure 8a, b and c Pipe Installation and Placement

The first pipe was carried down the tunnel to the starting station and positioned against an I-beam in order to restrain movement of the pipe during placement of subsequent pipe, Figure 9a. Additional pipe installation began with stabbing the incoming pipe onto the previously installed pipe utilizing a butt weld with steel backing then adjusting the pipe to the proper line and grade. During this process the butt joint was tack welded or root welded to provide a temporary connection. After adjusting to the final location and performing the initial pipe joint fit-up the pipe was blocked to prevent further movement and secured for final fitting and welding operations, Figure 9 b and c.

Figure 9a, b and c Pipe Placed Against I-Beam, Blocking and Welding

Approximately 2000 feetor 50 pipe lengths were installed within the first 11 days on the expedited two shift schedulebeginning on 7-31-14. On 8-11-14 while in process and after installing the first 2000 feet,two pipe joints which were temporarily tacked/rooted began to pull apart in what appeared to be thermal contraction, Figure 10a, and b.Due to the serious nature of this event, numerous meetings followed and it was pointed out these joints acted to relieve the thermal stress within the pipe as it contracted and the suggestion was made to remove the tack welds on previously installed joints to allow for potential additional thermal contraction but this method was rejected as it would cause numerous pipe joints which would exceed the maximum root tolerance.Accordingly, temporary attachments were immediately used to restrain any further movement, unfinished root welds were immediately completed and additional supplemental weld metal was added to the remaining in process pipe joints, Figure 10c.

Figure 10a, b and c Pipe Pulling Apart and Temporary Attachments

Separation of the first two pipe joints amounted to an increase in root opening of approximately 1.5” on each joint which was cause for concern and required remedial work at the joints in order to meet the approved Welding Procedure Specification (WPS). A new WPS was prepared to correct the two joint deficiencies back to the approved WPS which was executed and the joints completed, Figure 11a, b and c.

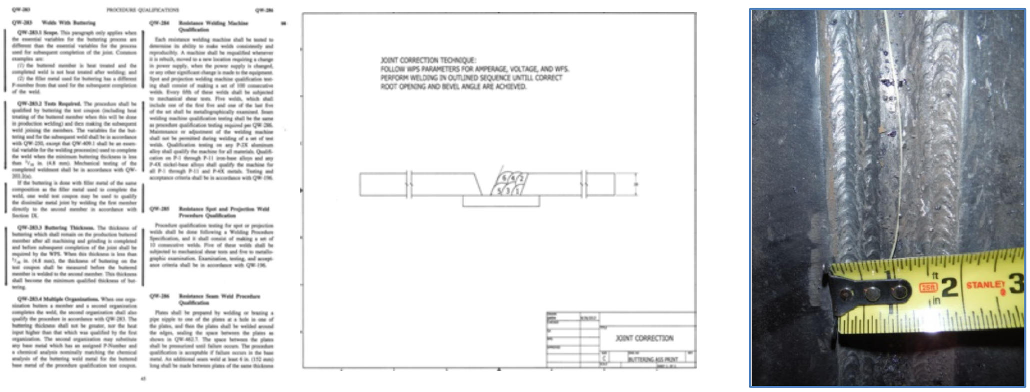

Figure 11a, b and c Remedial WPS and Completed Weld

Concurrently,further investigation determined there was a 34 degree Fahrenheit temperature differential between the pipe stored on the surface compared to the pipe installed in the tunnel pipe. This information is summarized below, Table 1.

DIGITAL INFRARED THERMOMENTER READINGS						
PERFORMED 8-12-14						
SURFACE PIPE				TUNNEL PIPE		
STEEL PIPE INVERT	118			STEEL PIPE INVERT	76.2	
STEEL PIPE CROWN	113			STEEL PIPE CROWN	86.5	
MORTAR INTERIOR SHADED	107			MORTAR INTERIOR INVERT	77.4	
MORTAR INTERIOR SUN EXPOSED	127			MORTAR INTERIOR CROWN	82.7	
PIPE EXTERIOR MORTAR SHADED	101			PIPE EXTERIOR MORTAR INVERT	80.9	
PIPE EXTERIOR MORTAR SUN	111			PIPE EXTERIOR MORTAR CROWN	86.3	
				TUNNEL WALL	74.3	
STEEL PIPE AVERAGE TEMPERATURE		115.5		STEEL PIPE AVERAGE TEMPERATURE		81.35
STEEL PIPE SURFACE/TUNNEL DELTA		**34.15**				
LOCAL AREA AVG TEMP HIGH/LOW	**85.14**	**62.86**				
LOCAL AREA TEMPERATURES 7 DAYS PRIOR TO INCIDENT; Aug 4 high 88 low 66, Aug 5 high 87 low 60, Aug 6 high 80 low 61, Aug 7 high 80 low 61, Aug 8 high 85 low 63, Aug 9 high 86 low 65, Aug 10 high 90 low 64. **AVG HIGHS 85**, **AVG LOWS 63**						
NOTE: PIPE TEMPERATURE ARE SHOWN IN DEGREES FAHRENHEIT. MEASUREMENTS WERE TAKEN ON SURFACE PIPE MARK 36 AND TUNNEL PIPE TEMPERATURES MEASUREMENTS WERE TAKEN ON MARK 38						

Table 1 Temperature Readings For Pipe and Air

After gathering information on the local conditions the temperature delta was entered into an equation for thermal expansion/contraction to evaluate the expected pipe contraction. The conclusion indicated there could be additional movement beyond that found in the initial two pipe joints, Figure 12a.

Linear Expansion/Contraction

$\Delta L = \alpha L_o \Delta T$

Given:

ΔT =34.15°F

L_o=24,000 inches

α=6.5x10^{-6}inch/(inch °F) Carbon Steel

So;

ΔL=5.33 inches

Linear Force Calculation

$\sigma = ExKx\Delta T$

Given

E=30x10^6 lbs/in^2

K=6.5x10^{-6} 1/°F

ΔT=34.15°F

So;

σ=6659.25 lbs/in^2

A=Πx98 inchesx9/16 inches=173.18 in^2

F=σxA

So;

F= 1,153,250 lbs

Figure 12a and b Linear Expansion/Contraction and Linear Force Calculations for Pipe

The thermal contraction event can be summarized with the expectation of 5.33 inches linear movement (contraction) in the 2000 feet of pipe which amounts to .11 inches per pipe length and utilizing the Linear Force Calculation we learn the anticipate force will amount to 1.1 million pounds of force, Figure 12b. Which are both well beyond the design of the support blocks.

As indicated by the calculations, and while addressing a plan of action, the same 50 joints continued to contract for an additional 8 days and on 8-19-14 virtually all 2000 feet of the installed pipe had contracted to the point of toppling the support blocking thereby dropping the tunnel liner several inches within the tunnel, Figure 13a and b. The pipe had also contracted away from the I-beam (Figure 9a) placed against the first pipe installed by 3.78 inches.

Figure 13a and bToppled Support Blocking

The axial load was enough to deform a 3" x 3" x .25" square washer and pull the support block steel bolts completely through the block, Figure 13a. Ironically, other than changing the carrier pipe elevation within the tunnel, the only detectable damage to the steel pipe was a deformation found in last pipe installed, Figure 14a and b.

Figure 14a and b Pipe Elevation Change and Pipe Deformation

The remaining 1000' of tunnel liner was installed on a single shift basis with limited productivity to allow most of the pipe thermal contraction to occur during the slowed process. The pipe supports were doubled under each pipe for added measure and the remaining tunnel liner was installed without incident, Figure 15.

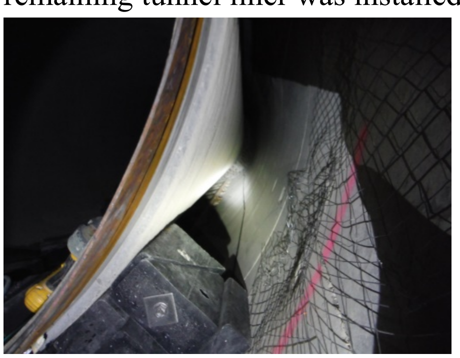

Figure 15 Double Support Blocks

THERMAL CONTRACTION LESSONS LEARNED

1) The extreme temperature differentials from the offsite pipe storage to the lower tunnel temperature would normally be moderated during the overnight stay in the cooler job site location. However the heavy exterior mortar coating and inner tape wrapping acted to insulate the pipe and delayed cooling of the pipesteel core for at least 19 days (7-31-14 to 8-19-14).
2) AWWA C206 states "Anticipated thermal stresses should be evaluated by the purchaser" and AWWA M-11 recommends the use of special closure joints for lap joint pipe, which leaves an un-welded joint every 400-500 feet distance to act as an expansion/contraction joint. After the pipe leading to and from this closure joint has been allowed to cool the closure joint can then be welded. This same principle can be adapted to butt joints by utilizing the backing bar (assuming the joint tolerance accommodates this use) or leaving a short section of pipe out of the run until the pipe leading to and from this short section has cooled to ground temperature.
3) The use of attached or welded pipe supports are much more robust than independent blocking and can accommodate axial movement by yielding, however the mesh used for cathodic protection on this project could have been severely damaged by any axial movement of the pipe.
4) Manually cooling the pipe was considered for the remaining pipe however the use of shading was impractical because of the logistics in moving such a large quantity of pipe with a crane and the limited onsite storage area/time. Using water spray to cool the mortar coating was also considered but would add considerable weight to the pipe and introducing water into a welded joint would adversely affect the weld quality.
5) The use of FCAW (Flux Cored Arc Welding) out performs SMAW (Shielded Metal Arc Welding or Stick) by a factor of 2.5 to 3 times and will expedite completing the joint connection which would avoid the joint separation experienced, however this will have no direct effect on thermal contraction.
6) Conventional expansion joints can also be used to accommodate this condition but is not a common practice in tunnels due to the expense and inability to replace a soon to be concrete embedded joint.

CONCLUSION

Thermal contraction is a very critical issue on projects where the pipe temperature during installation is expected to differ substantially from the ground temperature or operating temperature. The location of a project, time of year and location of the incoming pipe/materials will all play a part in potential thermal issues. Proactively taking temperature measurements of these differentials can determine if there is actually an issue and the Specifications should provide a means to accommodate or correct the condition. The team on this project was very proactive and cooperative to investigate the cause and consider solutions. As a result of this cooperation by all parties the steel liner was ultimately completed on schedule despite the challenges.

REFERENCES Not applicable

An Engineer's Guide to Nondestructive Weld Examination

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Abstract

The application of nondestructive examination (NDE) for welding of steel pipe and steel cylinders is a topic that requires forethought and knowledge. Specifically, when to use a certain NDE method, how to determine the frequency of examination, and the acceptance criteria to apply. NDE methods often vary from project to project, and understanding each method will help the engineer determine which process or combination of processes to use for a specific project. These quality assurance checks are useful for confirming weld quality in the final product before it is put into service. A weld that looks great on the outside is often seen as high quality and this is hopefully the case. Unfortunately, surface appearance alone cannot be used to verify good workmanship or internal weld quality. Many pipeline owners operate on limited budgets and need systems that can perform for many years with minimal or no maintenance. Water main and other pipeline ruptures almost always make front page news, which is why following good design practices in combination with the use of secondary checks to verify fabrication quality are important. Most pipe fabrication standards require some NDE, typically in the form of hydrostatic testing and visual inspection. These tests are useful for locating leaks and surface defects, but they will not confirm that a weld has complete joint penetration or that individual weld beads are free from objectionable defects. Additional forms of NDE must be specified by the engineer and are often needed for critical applications and projects that involve field welding to confirm weld quality. This paper will provide an overview of Visual Inspection responsibilities and additional NDE methods specified by engineers. A description of what each NDE method is capable of detecting, its benefits and limitations is also included. Specific NDE methods that will be discussed include:

- Radiographic Testing (RT)
- Ultrasonic Testing (UT)
- Magnetic Particle Testing (MT)
- Liquid Penetrant Testing (PT)

VISUAL INSPECTION RESPONSIBILITIES

The first step is to understand fabricator and contractor responsibilities for welding inspection and quality control. Visual inspection begins at the shop with an examination of incoming materials, checks on joint preparation, root openings, and assessing the quality of alignment and cleanliness. Most pipe fabrication shops adhere to a quality control program that includes the American Society of Mechanical Engineers, Boiler and Pressure Vessel Code (ASME BPVC), Section IX *Welding, Brazing and Fusing Qualifications* for qualification of shop welding procedures and performance qualifications for welders.

The fabricator and contractor are responsible for visual inspection of shop and field welding, and ensure that the correct welding procedures are used by qualified welders. Shop welding typically complies with ASME BPVC, Section VIII, Div. 1, Part UW. Welding for pipe fabricated to meet American Water Works Association (AWWA) C200 *Steel Water Pipe, 6 In. (150 mm) and Larger*, must also comply with AWS D1.1 *Structural Welding Code – Steel*, Table 6.1. The welding inspector oversees a few runs, paying particular attention to the root passes, which is where cracks often initiate.

Workmanship techniques are monitored to ensure that subsequent weld passes are run at the correct speed with proper current settings and arc lengths. Changes to any of these can result in elongated ripples, spatter, and undercut, inadequate joint penetration, porosity or slag inclusions. Once the welder finishes the joint, the welding inspector then performs a final visual inspection of the entire weld. As can be seen by the discussion above, visual inspection is a multi-step process requiring the welding inspector to be present from start to finish and it is an essential part of fabrication quality control. Unfortunately there have been instances when the welding inspector arrives in the shop, or at the site, after the welding has been completed. The result is an incomplete visual inspection that does not meet welding code requirements.

Aside from visual inspection, most pipe standards also include requirements for shop hydrostatic testing. While this is a useful and desired test, it does not confirm much about weld quality aside from the fact that the welds are not currently leaking. Additionally, hydrotest pressures are sometimes held for as little as two seconds, hardly enough time for an inspector to check the entire length of weld on a pipe stick to confirm no leakage. The engineer can specify that the hydrotest pressure be held for a minimum of five minutes, or longer if needed so that the inspector can visually examine the entire weld length.

A summary of the contractor's inspection responsibilities is provided below:

> **Contractor's Inspection** Visual inspection and correction of deficiencies in materials and workmanship necessary to meet welding code and contract requirements. This includes monitoring of manufacturing methods so that adjustments and corrections can be made during the fabrication process if defects are identified.

SPECIAL INSPECTION

Aside from the nondestructive testing described above that is always the responsibility of the fabricator and contractor, any additional weld examination or testing must be specified in the contract documents by the Engineer. A common misconception is that if a pipe standard such as AWWA C200 is part of the contract requirements, all NDE needed will automatically be provided as part of the fabrication work. It is important to note that the additional testing and NDE discussed below will not be provided unless it is specified by the engineer in the contract documents.

NDE specified by the engineer is often referred to as Verification Inspection or Special Inspection and includes methods such as RT, UT, MT and PT. When performed at the shop, many contracts will require this NDE to be part of the fabricator's responsibilities, with the additional cost borne by the fabricator. Alternatively, the NDE could be done by a special inspection agency employed by the Owner. Special inspection for field welding is almost always provided by the Owner's special inspection firm. These NDE methods are discussed in the following sections and a summary of Special and Verification Inspection is provided below:

> **Special and Verification Inspection** Testing and inspection done at the Engineer's or Owner's request to provide confidence that defects are not present in the final product. Examples include dye penetrant, magnetic particle, radiographic and ultrasonic testing. NDE methods, frequency of examination and acceptance criteria are specified by the Engineer and included in the contract documents.

NONDESTRUCTIVE TESTING METHODS

The following sections provide a description of various NDE methods, including its uses, limitations, acceptance criteria and a suggested testing frequency for typical projects. More stringent applications may require testing at a more frequent rate.

Radiographic Testing (RT)

- Description: RT is used to verify that groove welds do not have internal defects and to verify the depth of weld penetration. This method involves the use of a radioactive source to take an x-ray of the weld cross-section, and is often preferred by engineers as it produces a permanent record of the test and weld cross section on film. RT must be performed by an NDE technician certified in accordance with the American Society for Nondestructive Testing (ASNT), *Recommended Practice No. SNT-TC-1A* as RT Level II.

- Limitations: RT results for butt joints with backing (see Figure 1) are more difficult to interpret as the interface at the backing can produce false readings. RT is not an appropriate test method for T-joint groove welds or for fillet welds.
- Suggested Frequency and Acceptance Criteria:

- Shop: For shop welding, a very minimal RT inspection requirement is spot RT per ASME BPVC Sec. VIII, Div. 1, Paragraph UW-52. Spot RT is a 1% inspection frequency, which translates to RT of 6 inches of weld every 50 feet of weld. ASCE Steel Penstocks, Section 3 provides additional recommendations regarding inspection frequency.

- Field: The frequency of field RT is a function of the expected stress levels, fatigue and service life of the weld. The difficulty in making the weld due to accessibility and field conditions should also be considered. Welds made from one side without backing or backgouging require higher welder skill and are more apt to have defects within the root pass. A typical RT inspection frequency for butt joint groove welds is 10% random RT, specified to meet AWS D1.1, Paragraph 6.12.1.

<u>Ultrasonic Testing (UT)</u>

- Description: UT can be used for examination of all groove welds, although it is typically specified in the shop only for cases when RT should not be used (for example at outlet or tee connections). This method involves the use of an ultrasonic transducer and a couplant. In the field, UT is sometimes needed as it does not involve the use of a radioactive source and therefore the immediate area doesn't need to be cleared of other personnel prior to testing. UT is also used to check base metal quality prior to fabrication as it can detect laminations within steel plates.

  A relatively new UT device is Phased-Array UT (PAUT), which can be used with a digital recorder to produce a permanent record of the test. UT and PAUT technicians must be certified in accordance with SNT-TC-1A as Level II.
- Limitations: Generally not applicable for materials less than 5/16 inches in thickness. Highly dependent on the UT operator skill level. Does not produce a permanent record of the weld cross-section, unless PAUT is used with a digital recorder.
- Suggested Frequency and Acceptance Criteria:

- Shop UT: For shop welding, UT inspection is typically used only when RT cannot be used. In this case, 100% UT inspection in accordance with ASME BPVC Sec. VIII, Div. 1, Paragraph UW-53 is recommended. ASCE Steel Penstocks, Section 3 provides additional recommendations regarding inspection frequency.
- Field UT: Similar to field RT, the frequency of field UT is a function of the expected stress levels, fatigue and service life of the weld. The difficulty in making the weld due to accessibility and field conditions is to be considered. A typical UT inspection frequency for field welds is 10% random UT, specified to AWS D1.1, Paragraph 6.13.1.

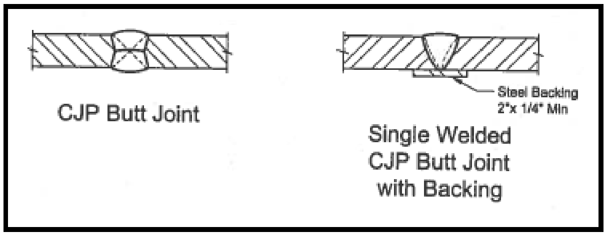

Figure 1: Common Groove Welds (ASCE Steel Penstocks)

Magnetic Particle Testing (MT)

- Description: MT is essentially an enhancement of visual inspection and is used for fillet welds and partial joint penetration welds (see Figures 2 and 3), and to inspect weld surfaces and root passes to verify the absence of surface defects. MT can also detect some near-surface discontinuities. This method involves the use of a yoke or other device that creates a magnetic field in the area being examined. Magnetic particles are then sprinkled over the area and will migrate to any crack or discontinuity, making it more visible. Technicians must be certified in accordance with SNT-TC-1A as MT Level II.

- Limitations: MT cannot be used on stainless steel, or on other non-magnetic materials.

- Suggested Frequency and Acceptance Criteria:

- Shop MT: For shop welding, 100% MT inspection is typical requirement for fillet welds. Acceptance criteria is the same as the acceptance criteria for shop VT (ASME BPVC SEC VIII, Div. 1, Section UW).
- Field MT: 20% random MT inspection is typical for field welds. Acceptance criteria is the same as the acceptance criteria for field VT (AWS D1.1, paragraph 6.10).

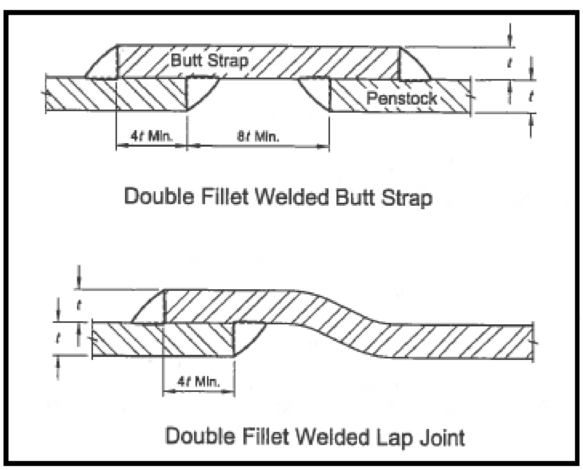

Figure 2: Typical Fillet Welds (ASCE Steel Penstocks)

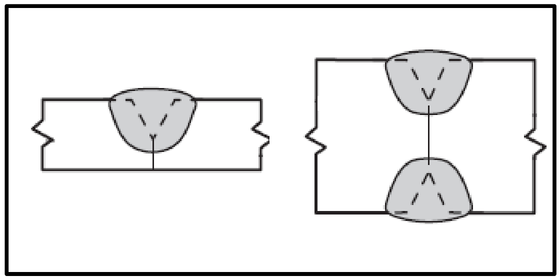

Figure 3: Typical Partial Joint Penetration Welds

Liquid Penetrant Testing (PT)

- Description: PT is similar to MT in that it is essentially an enhancement of visual inspection and is used for fillet welds, partial joint penetration welds, and to inspect weld surfaces to verify the absence of surface defects. A penetrant is applied to the area to be inspected followed by a developer, making any areas with discontinuities more visible. Technicians must be certified in accordance with SNT-TC-1A as PT Level II.
- Limitations: Surface coatings may hide defects.
- Suggested Frequency and Acceptance Criteria:

- Shop PT: For shop welding, 100% PT inspection is typical requirement for fillet welds. Acceptance criteria is the same as the acceptance criteria for shop VT (ASME BPVC SEC VIII, Div. 1, Section UW).
- Field PT: 20% random PT inspection is typical for field welds. Acceptance criteria is the same as the acceptance criteria for field VT (AWS D1.1, paragraph 6.10).

SUMMARY

Understanding nondestructive testing methods and applying them correctly as secondary checks of weld quality is a good way to evaluate fabrication quality. Highly stressed or safety critical welds often will require more frequent examination. However even piping systems operating at low pressures may be subjected to unanticipated stresses at welded joints due to longitudinal bending, lateral loads, thermal stresses and pressure transients. So even though the pipe wall thickness may be more than adequate, if the welds have defects the system may fail.

NDE type, frequency and acceptance criteria must be specified by the engineer in the contract documents. This NDE is in addition to the 100% Visual Inspection completed by the contractor’s welding inspector, and any other NDE required by referenced pipe standards. Appropriate NDE methods for the different weld types are summarized in Table 1 below:

Weld Type	NDE Method(s)
Butt joint groove weld without backing	RT or UT
Butt joint groove weld with backing	UT
Tee joint groove weld	UT
Partial joint penetration groove weld	MT or PT
Fillet weld	MT or PT

Table 1: Welded Joint Types and NDE Methods

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Streamlining the Submittal Process—Do's and Don'ts

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Abstract

This paper discusses several recommendations for preparing, reviewing, and responding to contractor's submittals for water and wastewater conveyance projects. These conveyance projects typically involve submittals for pipe materials, pipe fittings, pipe joints, pipe lining, pipe coating, welding, manholes, valves, and pumps. The paper includes perspectives from the supplier's, contractor's, and designer's involvement in the process. At times, the submittal process can be time-consuming and frustrating for all participants. Suggestions are presented to improve preparation of submittals, to speed up the processing of submittals, to minimize re-submittals, and to generally streamline the submittal process.

INTRODUCTION

The requirements for contractor submittals are generally described in the project specifications. Frequently, a section titled "Submittal Procedures" is included in the General Requirements division. This section defines how the contractor's submittals are to be transmitted, the time allowed for the owner's or engineer's review of the submittals, the possible dispositions, and the contractor's actions to be taken for each disposition. Specific submittal documents are frequently required in the technical specification sections. For example, the section for pipe typically requires a submittal identifying the proposed pipe dimensions, joint details, and pressure class. The section for disinfection of a pipeline typically requires require a submittal consisting of the certified bacteriological test results.

SUBMITTAL TYPES

Some specifications differentiate between "Action" submittals and "Informational" submittals. Action submittals include items such as detailed shop drawings, manufacturer's literature, and other items that require review and approval by the engineer or owner before the contractors and suppliers can proceed with manufacturing or furnishing the item being proposed. Informational submittals include such items as certificates of proper installation, affidavits of compliance, test results, and O&M manuals which do not require formal approval in order to proceed with the proposed work.

SUBMITTAL DISPOSITIONS

The choice of submittal dispositions varies depending on the owner's or designer's preference. The most common dispositions include:

- Approved
- Approved as Noted
- Partial Approval, Resubmit as Noted
- Revise and Resubmit
- Rejected
- Not Subject to Review

If the disposition is "Approved" or "Approved as Noted", the contractor may proceed with the work covered by the submittal in accordance with the engineer's or owner's notations. If the disposition is "Partial Approval, Resubmit as Noted", the contractor may begin to incorporate the products or implement the work, with the exception of the portions requiring resubmittal. If the disposition is "Revise and Resubmit" or "Rejected", the work should not proceed until the submittal has been resubmitted and approved. Some documents, such as shoring plans designed and stamped by the contractor's engineer, are not subject to the design engineer's review, but it is important for the owner and the owner's engineer to know that the document exists and is filed in the project records.

GENERAL GUIDELINES FOR SELECTING DISPOSITIONS

If the reviewer wants to confirm that a change has been made, they should mark the submittal "Partial Approval – Resubmit as Noted" rather than "Approved as Noted". The "Approved as Noted" disposition does not require any confirmation that the change has been made. Examples of submittals where a partial resubmittal is appropriate include a submittal where a pipe or valve diameter is incorrect or a weld detail is shown incorrectly.

If a submittal includes two or more items, and the disposition of "Partial Approval – Resubmit as Noted" is not allowed by the owner, consider stamping the portion of the submittal that needs no corrections with "Approved" and stamping the remainder with "Revise and Resubmit". This allows production and installation of the correct items to proceed, and can save the project both time and cost.

If a correction is "minor" in nature, such as revising a dimension that is obviously incorrect, consider marking the submittal "Approved as Noted" and note the correct dimension. Other examples of "minor" corrections include adding a note saying an item must be field verified prior to fabrication, or noting that a valve must be furnished with a handwheel when the submittal shows a drawing of a valve with a 2-inch operating nut.

Examples of submittals where the dispositions of "Revise and Resubmit" or

“Rejected” are appropriate include items such as showing a Class 150 valve when a Class 250 valve is required by the specifications, or showing a restraining device with set screws when a device with wedges is required.

A disposition of “Not Subject to Review” is appropriate for submittals showing contractor-designed items such as a temporary power supply system or a temporary support system for an existing pipe over an open trench, or items where the responsibility is clearly the contractor’s, such as health and safety plans. Other contractor-designed facilities such as a bypassing plan are frequently reviewed by the owner/engineer because it is important that these facilities are sized correctly for the anticipated flows and that suitable back-up facilities are provided.

PROCESSING OF SUBMITTALS

Submittals are generally numbered sequentially for record keeping. Many owners/engineers use zeros as part of the submittal number in order to keep the number of digits consistent (such as 002, 045, etc.). This allows the submittals to be more easily sorted by number with various software programs. The documents are frequently entered into a submittal log for tracking. When a resubmittal is received, a letter is generally added to the original submittal number (002A). Subsequent resubmittals for the same submittal receive the next letter (002B). When partial resubmittals are received, the same numbering system (i.e. adding a letter after the submittal number) is generally followed.

The document control procedures for some projects require that all transmittals be numbered sequentially, including submittals. This can be confusing, as transmittal numbers are frequently different than submittal numbers. Similarly, a submittal from a subcontractor may be identified as No. 001 in the subcontractor’s system, whereas it may be assigned a different number in the general contractor’s system.

Some submittals may require review by more than one of the members on the designer team. For example, a submittal may include information that should be reviewed by a pipeline designer, a corrosion engineer, and by a structural engineer. A person familiar with the responsibilities of the design team should determine which staff members are appropriate to review each submittal. Routing slips should be attached to submittals, so each reviewer knows who will be performing each part of the review. Designers performing the review should be actively involved in the assignment of additional reviewers. Occasionally, it is important for other designers to be aware of all the comments that are made. For example, if the pipeline engineer notes that the dimensions of a wall penetration are incorrect, the structural engineer may need to comment on the reinforcing in the wall. Open and active communication is critical to submittal reviews. Frequently, the project manager or construction services manager will check the submittal responses received from his/her staff to make sure that the appropriate designers have reviewed the submittal and coordinated their responses. The project manager or their designee is also generally responsible to make sure the submittals are returned on time.

DO's AND DON'Ts FOR PREPARATION OF SUBMITTALS

This section includes several recommendations regarding preparation of submittals by contractors, subcontractors, and material suppliers.

- **DON'T** submit pages from a catalog that include several unmarked choices (such as a pipe catalog that shows several optional pipe classes and diameters). If the contract requires 24-inch diameter, Class 250 pipe, mark the size and class that is proposed to be furnished.

- If the specifications state that a product shall be NSF certified for potable water, shall be rated for HS-20 loading, or similar requirements, **DO** make the effort to ensure that all the requirements stated in the specifications are addressed in the submittal. Not addressing all the requirements is one of the more frequent causes for submittals to be rejected, revised, and re-processed.

- If the drawings require that an item (such as an existing pipe outside diameter, an existing pipe slope, or an existing pipe elevation) be field verified, **DO** indicate on the submittal that the item has been verified and that the submittal is based on the verified information.

- If a supplier needs to have an item confirmed, and the submittal includes a callout requesting confirmation as shown in Figure 1 below, **DO** consider identifying who is being asked to make the confirmation (i.e. is the request addressed to the designer, to the general contractor, or to another material supplier?). Occasionally, no one responds to the request because each party assumes the other party will respond.

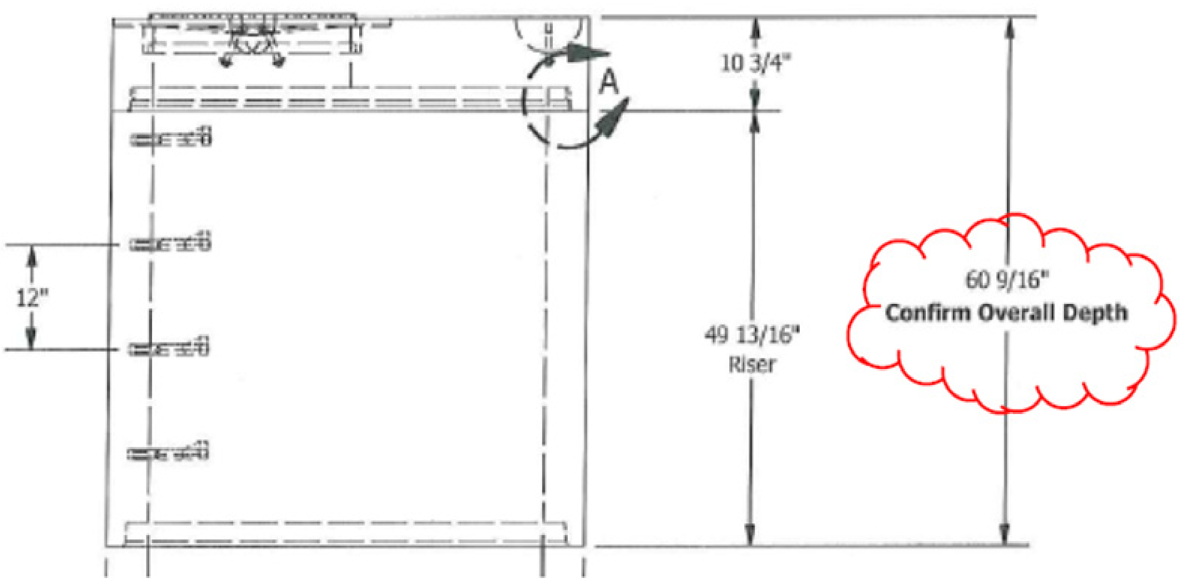

Figure 1. Information Request

- If the equipment or materials being submitted do not comply with the specifications and a variance to the specifications is proposed, **DO** make it clear that a variance is being requested. Some transmittal forms have a box that can be checked to indicate whether or not the submitted items meet the requirements of the specifications. Bringing a variance request to the reviewer's attention can avoid misunderstandings as the project's construction proceeds.

- If a material supplier requests a variance that impacts other submittals, and the variance is approved, **DO** make the effort to coordinate and revise all the affected submittals. For example, if a flange drilling pattern of a valve is changed, coordinate the change with the pipe supplier who will be providing the matching flange. If a pipe outside diameter is changed, coordinate the change with the supplier who will be providing the manholes.

- If the specifications require that calculations submitted by the contractor be stamped by a professional engineer, **DO** arrange for the calculations to be stamped. If the specifications require that the professional engineer who stamps the submittal be registered in the state where the work is being constructed, confirm that the engineer is registered in that state.

- If it becomes apparent that an approved product cannot be delivered in a timely manner, and another supplier's product is available, **DO** make it clear on the subsequent submittal that this submittal is intended to replace an item that has been previously approved, and that the previous submittal should be withdrawn or removed from the files.

- If the transmittal form does not have a blank line for the specification section where an item is specified, **DO** indicate this information somewhere on the form (e.g. Section 33 05.01, Article 2.06.B). Adding this information can prevent confusion for the reviewers, particularly if a similar (but different) product is specified in another specification section.

- Similarly, if a product included in a submittal is appropriate at only certain locations in the project, **DO** note this information on the transmittal page (e.g. state that "This Class 150 valve is intended for use only at Station 46+23. All other valves will be Class 250.").

- If quick processing of certain submittals is necessary to maintain a project schedule, **DO** mark the submittals as "URGENT" or "HOT". While this does not guarantee speedy review and processing, it does improve the likelihood that the submittal will be returned quickly.

- If several items are being submitted concurrently, **DO** consider using separate submittal numbers for each one. If only one of the items requires a re-submittal, the other items can be returned as "Approved".

DO's AND DON'Ts FOR REVIEWING SUBMITTALS

This section includes several recommendations regarding reviewing and responding to submittals by design engineers and owners.

- If an item in a submittal appears incorrect, **DON'T** just circle the item and add a question mark. **DO** make the effort to indicate why the item appears to be incorrect. Frequently, contractors or suppliers will not be able to read the mind of the reviewer and understand why the item is being questioned.

- **DON'T** return submittals with a vague letter addressing the issues. **DO** return the submittals with clear and specific indications of any discrepancies.

- **DO** respond to submittals by either returning the full submittal, or a letter with a reference to the contents of the submittal. For example, in a letter consider stating "Submittal xyz consisting of Drawings 1, 2, and 3 and welding procedure WP-MIG139 is approved with no exceptions".

- When reviewing a submittal, **DO** plan to include all your comments in the first response. **DON'T** reject a submittal with a few initial comments with the intent of spending more time reviewing it when it is returned.

- If the disposition "Partial Approval – Resubmit as Noted" is used, **DO** be clear regarding exactly what needs to be resubmitted. For example, indicate which pages of the document need to be resubmitted and why. For the contractor's reference, consider providing the specification section and article number which identifies the requirement that was omitted.

- While performing a submittal review, **DON'T** submit to the temptation to make design changes or improvements not shown in the bid documents. If a change is necessary, follow the established procedure to initiate design changes, such as initiation of a Work Change Directive.

- **DO** make an effort to return submittals in a timely manner. Submittals returned before the due date are always appreciated by the contractor and suppliers. This is particularly true if the disposition requires a re-submittal. For a critical item, returning a submittal a day or two earlier can make a huge difference in the project budget and overall project success.

- If the information in the submittal is outside a reviewer's area of expertise and there is uncertainty if an item needs to be revised, **DO** either consult with someone who understands the subject matter or contact the contractor or supplier for clarification before choosing a disposition of "Revise and Resubmit".

- If the corrections to be made impact only a portion of the submittal, **DO** consider stamping the sheets needing correction as "Revise and Resubmit", and stamp the remainder of the sheets "Approved". This allow fabrication or installation of the approved items to commence.

- If a submittal includes a request for the designer/reviewer to confirm a dimension, size, or location, **DO** make the effort to provide the requested information as appropriate. If the request should be answered by the owner or general contractor, **DO** consider adding a note identifying who should respond. **DON'T** just ignore these requests, as this will likely delay the submittal process and perhaps require a resubmittal.

- If the submittal includes a dimension or elevation calculated by the contractor or supplier, and the reviewer calculates a different dimension, **DO** note the discrepancy on the submittal. Some reviewers will request that the person who prepared the submittal provide calculations showing how the dimension was calculated, as illustrated in Figure 2 below. Other reviewers choose to provide their calculations, and instruct the contractor or supplier to modify the dimension accordingly. Use caution with the latter approach, as the reviewer may not be aware of changed field conditions. Note that if the reviewer's calculations are incorrect, he/she bears some responsibility for the outcome.

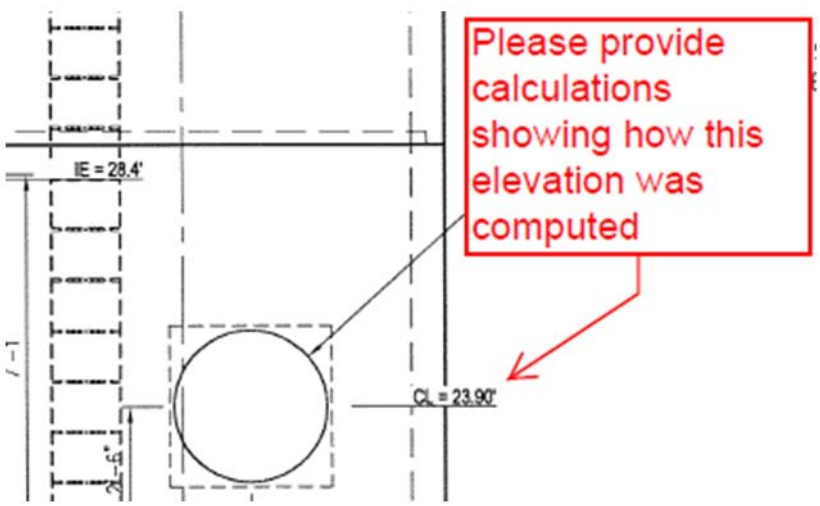

Figure 2. Request for Calculations

- Frequently, subcontractors will number their submittals sequentially. The general contractor often will assign a different number to the submittal based on their numbering system. **DO** use caution when filing submittals or referring to a submittal number. The number assigned by the general contractor is the number that should be used for filing and referencing the submittal.

ADDITIONAL RECOMMENDATIONS FOR STREAMLINING THE SUBMITTAL PROCESS

This section includes additional recommendations for streamlining the submittal process and improving communication between the various parties.

- During the project kick-off meeting or project chartering meeting, **DO** consider initiating a discussion of the procedures to improve communication and minimize re-submittals. For example, if a reviewer has a question regarding a submittal from a supplier, is it appropriate for him/her to call the supplier directly and ask the question? The alternative to a quick phone call may involve several steps: 1) noting the question on the submittal, 2) transmitting it to the owner's construction manager, 3) transmitting it to the general contractor, 4) transmitting it to the subcontractor, 5) transmitting it to the material supplier, 6) the supplier answering the question, and 7) the supplier returning the submittal through the same chain. This long processing chain requires time and effort from several staff and generally delays approval of the submittal.

- During the project kick-off meeting, **DO** consider discussing which of the products to be furnished have long lead times, and which submittals must be processed early in the project in order to meet the schedule.

- Some pipe suppliers have software to assist with the calculations for determining the elevation at the ends of each pipe length, the slope between grade breaks, the coordinates at the ends of each pipe length, and the angle of combined bends. If a reviewer is confident that the software provides accurate data, **DO** consider checking only the input data for the software program, rather than the output data. This can significantly speed up the review process.

- At the beginning of a project, **DO** provide clear instructions regarding the forms that must be used and what must be included on the cover page.

- **DO** consider using a transmittal form with the date clearly marked showing when the submittal is due back to the contractor. If the submittal process involves several steps (e.g. the reviewer returns the submittal to a project assistant, the project assistant forwards it to the owner, and the owner or construction manager forwards it to the contractor), allow adequate time for the process to unfold, and make it clear to the reviewer when he/she needs to have the review completed.

- Similarly, **DON'T** delay the submittal process by not forwarding the submittal in a timely manner. No one appreciates receiving a submittal to review the day before it is due.

- In general, **DO** avoid sending a submittal concurrently to two or more reviewers. Each reviewer may decide to wait until the other reviewers make their comments, or the comments received from the reviewers may not be coordinated and may even conflict. If the urgency of the submittal process requires concurrent reviews, **DO** give clear instructions to the reviewers regarding which portion of the submittal requires their review and how the coordination process is intended to occur.

- If the owner's operations and maintenance staff has specific preferences, **DO** consider involving them in the review process. Ideally, the O&M staff would have been involved in the design process as well. However, on one project, a certain type of manhole ladder was submitted and approved by both the designer and the owner's representative. During construction, a member of the owner's operation staff determined that the type of ladder being installed was not acceptable to his group. Replacing the ladders resulted in considerable cost increases, and could have been avoided at the submittal stage.

- Similarly, a certain valve or equipment manufacturer may be preferred by the owner's O&M staff. The local vendor for this product may have a reputation for prompt service, availability of spare parts, and product knowledge. The product may have a history of minimal required repairs, long life, ease of maintenance, etc. Ideally, the specifications should have stated "Valve shall be as manufactured by xxx. No substitutions will be allowed." However, occasionally when the submittals arrive, certain staff may begin to express their preferences. **DON'T** reject a submittal for a product that meets the requirements of the specifications based on a late-arriving expression of preference. Any changes to the requirements of the contract documents should follow the standard procedures, such as initiation of a Work Change Directive.

- As the owner or designer, **DON'T** attempt to assign submittal numbers prior to the award of the project construction contract. The submittal numbers should be assigned by the contractor as the work proceeds. On one project, the owner's representative attempted to assign numbers to all the required submittals based on the sequence of where they were mentioned in the specifications. This attempt to assign numbers ended poorly, as the first documents submitted by the contractor were based on what products were needed to start construction and which products had the longest production time, rather than in the sequence they appeared in the specifications.

- Occasionally, specifications describing a product will list two or three acceptable manufacturers and model numbers, and then state "or equal". When the contractor submits a product considered to be an equal, the reviewer needs to make an evaluation to determine if the product is, in fact, equal to the ones listed. An internet search of the words "or equal in specifications" will result in numerous documents with recommendations regarding how to evaluate if a product is an equal. As a reviewer, **DO** check the definition of "or equal" in the bid documents and **DO** make the effort to perform a careful evaluation of the submitted product.

- **DO** confirm that someone on the owner's team is assigned to verify that all the required submittals are completed. On some projects, the owner's team simply reviewed whatever the contractor submitted, and no one checked to confirm that all the submittals mentioned in the contract documents were received.

- Similarly, **DO** confirm that the inspection staff is aware of the status of the submittals. On one project, valves and other items were installed prior to the submittal being approved. When questioned regarding the reason, the owner's inspection staff stated that their workload did not allow them time to read and process all the submittal information that they received.

- As the project draws to a close, **DO** save files with the submittal information. Some owners do not keep the submittal information, and it is not available to staff in the future who are planning system modifications or improvements. Frequently, the design drawings do not show adequate details regarding pipe fittings, valve classes, direction of bells, and location of joints. The submittals often contain valuable information for future use.

- If submittals on a project are consistently poorly prepared and coordinated, **DO** consider alerting the appropriate contractor's staff. The person who prepares the submittals may be overloaded with responsibilities. On one project, after the contractor was alerted to the problem, a response was received saying essentially, "We acknowledge that recent submittals from subcontractors and suppliers have not met either your or our standards and are working to resolve the problem. We are aware that several resubmittals have been required and that these necessitated extra processing time by your staff and our staff." The care and quality of subsequent submittals was significantly improved after the exchange of messages.

CONCLUSION

Submittal preparation and reviews are an important part of almost every conveyance construction project. The submittals provide specific, detailed information regarding the proposed components of the project as well as what products were actually installed. If the submittals are poorly prepared or are lacking information, the submittal process can be slow and frustrating for the reviewers. Similarly, if the review comments are not clear or complete, the process can be frustrating for contractors and suppliers. Several recommended DO's and DON'Ts were presented in the paper for streamlining and improving the submittal process.

Liquefaction-Induced Differential Settlement and Resulting Loading and Structural Analysis of Buried Steel and Cast Iron Pipelines

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Abstract

The East Bay Municipal Utility District (EBMUD) is a major water utility providing water to over 1.3 million people on the eastern side of the San Francisco Bay Area. EBMUD has approximately 4,200 miles of treated water distribution and transmission pipelines within a 332 square-mile customer service area. System data are managed using a comprehensive Geographic Information System (GIS) geodatabase of pipeline characteristics, spatial location, and seismic hazards involving liquefaction, landslide, and fault crossing severities. Due to the proximity of active faults such as the Hayward, Calaveras, San Andreas, and Concord Faults, the EBMUD service area is prone to earthquakes and susceptible to secondary effects, such as liquefaction settlement, landsliding, and other permanent ground deformations. Previous studies (Prashar et al., 2012, and 2014) documented the susceptibility of these secondary effects. Permanent ground deformation was estimated from liquefaction induced settlements. The magnitude and spatial distribution of differential settlement were established in a GIS database for areas of Alameda and Oakland. Significant damage was predicted in all pipelines intersecting and also within:1) Artificial Fill, 2) Merritt Sands, 3) Holocene Alluvial Fan Deposits, and 4) Pleistocene Alluvial Fan Deposits. This paper summarizes the results of a pipeline structural analysis to identify pipeline segments with the potential for failure due to liquefaction induced settlement, using GIS. A detailed pipeline structural analysis was performed for the varying pipeline characteristics including diameter, length, pipe material type, and joint connection to evaluate the relative settlement estimates for four distinct soil deposit and geologic formation types. The goal of the study was to determine which pipelines would most likely fail in the event of an earthquake, with future steps to develop a plan to replace, reinforce, line, or install flexible connections at strategic locations of liquefaction induced differential settlement.

INTRODUCTION

The East Bay Municipal Utility District (EBMUD) service area is located in a highly active seismic area in the San Francisco East Bay Area. The Hayward Fault, which crosses the EBMUD service area, is capable of M7.0 earthquakes with a 140-year major event return cycle. The fault has a maximum earthquake of M7.25 with several thousand year return period. With the last major earthquake of M ~ 6.8 occurring in Hayward in 1868 (147 years ago), the next major quake is due at any time (USGS, 1993). The economic losses from a similar earthquake occurring today would likely exceed $165 billion in damages (Brocher, 2008) in the Bay Area.

Damage prediction models offer guidance for emergency response following an earthquake event. For a large-scale agency such as EBMUD that covers 332 square-miles of service area, damage can be geographically disbursed. Prediction models can support immediate triage efforts for inspection and response based on real-time data collection. For example, if model results indicate a potential for damage of a critical large diameter pipe, that pipe will be given higher priority for damage inspection over projected damage of a smaller diameter distribution pipe. Previous work by Prashar et. al., (2013) discusses the simplified approach to pipeline fragility to liquefaction and attempts to evaluate damage due to liquefaction. This paper discusses the structural analysis of the different pipeline types using the estimated liquefaction induced settlements for the areas of intersection boundaries detailed in Prashar et al., 2014 and identifies the pipelines which are most likely to fail in the event of an earthquake, so EBMUD can develop future steps on how to prevent by preemptive mitigation or respond and repair these pipelines.

BACKGROUND, HISTORY, AND GEOLOGY OF THE REGION

EBMUD was created in 1923 to provide water service in the San Francisco East Bay Area. In 1929, EBMUD started providing water to customers through the construction of the Mokelumne Aqueducts and the Pardee Dam. Today EBMUD provides water and wastewater treatment to 1.3 million users, including residential, industrial, commercial, institutional, and irrigation water users, in 20 incorporated cities and 15 unincorporated communities. EBMUD maintains 4,200 miles of pipeline infrastructure, 170 reservoirs, aqueducts, and tunnels. System data are managed using a comprehensive GIS geodatabase of pipeline characteristics, location, and seismic hazard.

Damage to water pipelines in areas with soil conditions poor in resisting earthquakes has occurred in the past. After the Loma Prieta earthquake in 1989, approximately 130 EBMUD water pipes in areas with geologic units of artificial fill and younger bay mud were repaired. Both cast iron and steel pipe material types shared a majority of the pipeline repairs following the earthquake. Today, some of EBMUD pipelines are located in areas with artificial fill, which are prone to liquefaction induced settlement and therefore, most likely to be damaged in the event of an earthquake. These areas include the cities of Alameda and Oakland, which have a high susceptibility to liquefaction because of their soil conditions, geologic conditions, shallow groundwater table, and are located close to seismic faults. In addition, liquefaction induced differential settlements and associated damage to pipelines has been documented from the 1993 Hokkaido Earthquake (Ling, 2003).

In a previous paper, 212 CPT soundings were evaluated using methods published in Holzer et. al. 2006, and the results of the evaluation showed that mostly artificial fill and some Quaternary Holocene alluvial fan deposits, located below the groundwater table, have a high probability of liquefaction under a Hayward M7.0 earthquake scenario. Figure 1 below shows the locations of the CPT soundings in the Alameda and Oakland areas (Areas 1 through 5) as well as the settlement and CPT results.

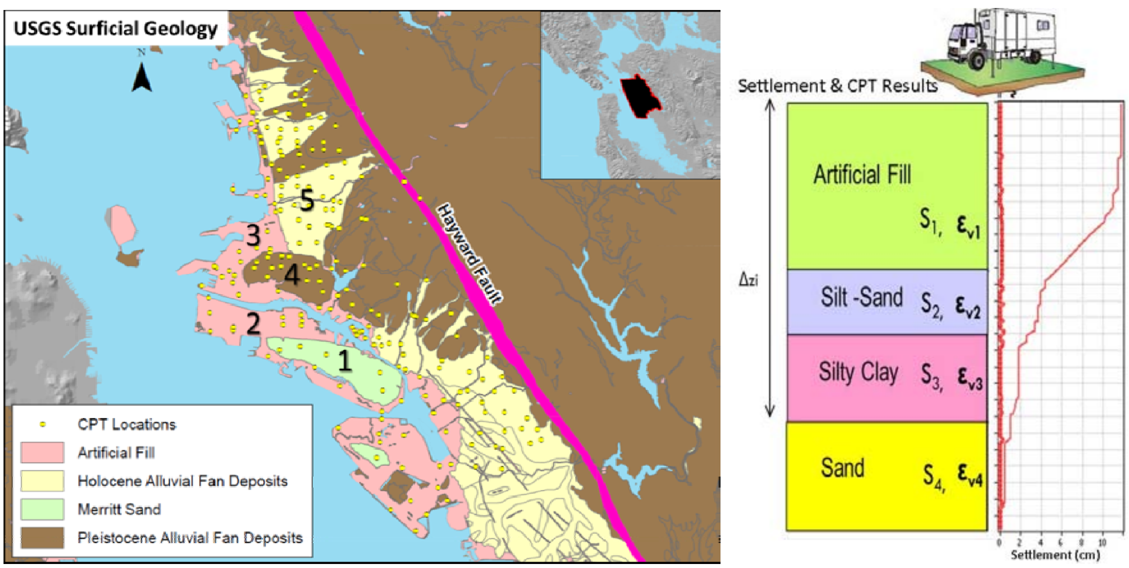

Figure 1: USGS Surficial Geology Zones and Settlement and CPT Results

APPROACH TO PIPELINE ASSESSMENT

Liquefaction induced settlements calculated and presented in a previous paper (Prashar et al., 2014) were used as a basis to evaluate pipeline structural response. GIS evaluations were performed to determine which pipelines in Areas 1 through 5 are most likely to be damaged due to liquefaction. Critical segments of pipes are located near the transition zones from a soft soil to a harder soil. The project team extracted pipeline segments from GIS to an excel spreadsheet, where the critical pipe length was determined by comparing deflection of the material with the soil deformation and pipe stress.

This assessment is of vital importance for EBMUD to clearly identify areas in its pipeline network that are unlikely to perform during a seismic event. In addition, the study will provide a better understanding of liquefaction induced settlement hazard and how it will affect the EBMUD water distribution capabilities. EBMUD's service area is located on an active seismic zone with a probability of occurrence of 63% for a moment magnitude M6.7 earthquake or greater (USGS, 2008). The approach taken to evaluate the potential damaging effects of an earthquake to the pipeline infrastructure of EBMUD will include the following steps:

1. The areas with the 212 CPT soundings by Holzer susceptible to liquefaction were classified into 5 general areas, shown in Figure 1, based on settlement estimates with PGA. GIS layers were used to map and form the boundaries between these areas (Prashar et al., 2014).
2. The relative settlements between the two adjacent areas for Areas 1 through 5 for different PGA were calculated and plotted for high, average, and low fragilities (Prashar et al., 2014).
3. The EBMUD GIS database contains information regarding the pipe network, pipe diameter, pipe material, length, and exact locations. The GIS database will help in the identification of segments of pipes in Areas 1 through 5 during a seismic event that exceed pipe material yield stress as discussed in the Pipe Structural Analysis section below. See Table 1 below for the pipes considered in this study.

Table 1: Breakdown of Pipe Type in Study Area

Material Type	Diameters (inches)	Miles of Pipe
Steel Welded Pipes (S)	4 – 36	295
Cast Iron Pipes (CI)	2 – 24	40

4. The steel and cast iron pipes were selected for structural analysis purposes since those pipe types were predominately in Areas 1 through 5, according to the GIS database.
5. The pipe’s bending stresses and strains were determined for various deflections or displacements. The steel pipe was analyzed as a fixed end beam at both ends with displacement at one end since it best represented the steel pipe crossing the boundary experiencing liquefaction induced settlement. The cast iron pipe was analyzed in a 10-foot segment as a cantilever beam with displacement at free end to represent the cast iron pipe crossing the boundary experiencing liquefaction induced settlement.
6. The deflections at yield bending stress for all steel and cast iron pipe diameters were calculated and compared to the relative settlements experienced by the pipes crossing the different boundaries.
7. The deflections at yield bending stress that exceeded the relative settlements were identified and GIS was used to create a map showing the critical zones where damage could occur during an earthquake event.

GIS AND RELATIVE SETTLEMENT ESTIMATES

Areas 1 through 5, shown in Figure 1, contain EBMUD pipes that were summarized by its material, diameter, and boundaries crossing. In this study, steel and cast iron pipes were considered, since these materials make up 81% of the pipes within the areas, as shown in Figure 2 below. For asbestos cement, wrought iron, and PVC pipelines, the project team assumed breaks during an earthquake event of large magnitude (+6). The number of steel and cast iron pipes by diameters in Areas 1 through 5 is shown in Figure 3 below. Nominal diameters of 8", 12", and 24" represent 81% of all steel pipes and nominal diameters of 4", 6", 8", 10", and 12" represent 96% of all cast iron pipes, which were used in the GIS analysis. The boundary between the areas in which the pipe crosses was assumed to be perpendicular with the ground surface.

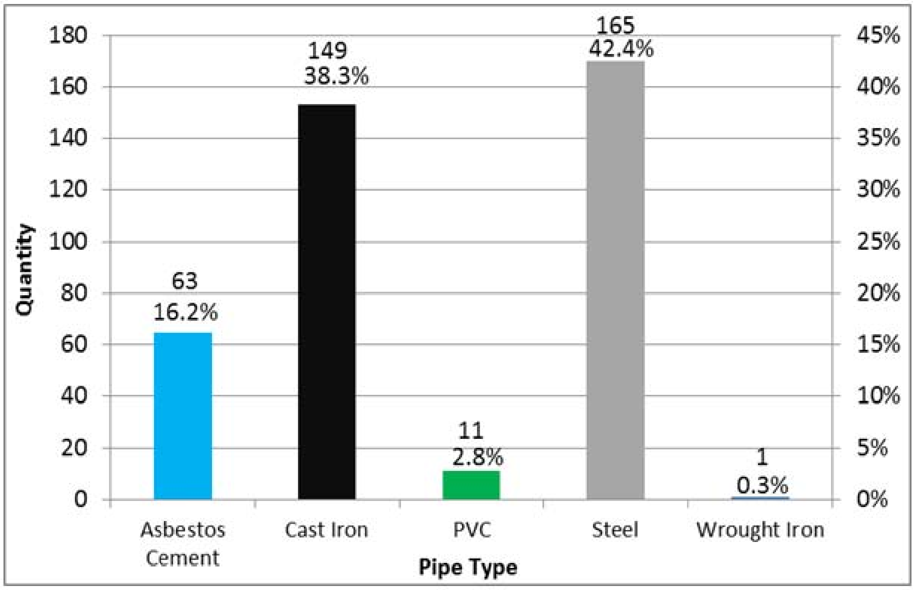

Figure 2: Pipes in Alameda and Oakland by Type

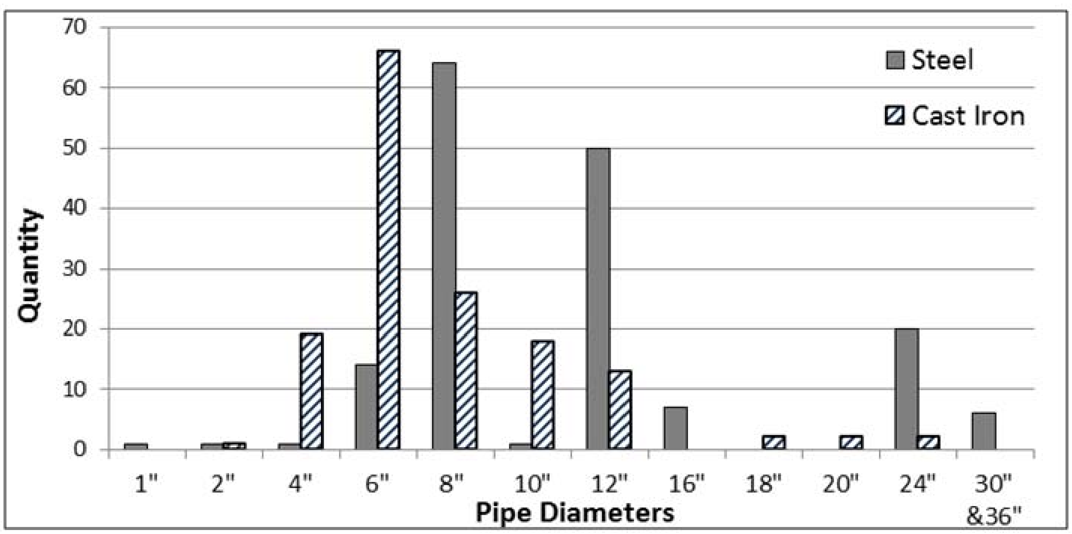

Figure 3: Steel and Cast Iron Pipes by Diameter

The relative settlement estimates for Areas 1 and 2, Areas 3 and 4, and Areas 3 and 5 for high, average, and low fragilities at PGA values from 0.1 g to 1.0 g (Prashar et al., 2014) were linked to corresponding pipes in those areas using Excel.

PIPE STRUCTURAL ANALYSIS

The key to correctly analyzing the pipe under bending is to understand the material properties and behavior of the steel and cast iron pipes. The cross section of a typical pipe is shown in Figure 4. The span is the length of pipe (L) considered in the study to evaluate differential settlement. Welded steel pipe can behave as a ductile material with consistent properties across the weld such that it can be considered to be a continuous pipe. Treating the steel pipe as a fixed end beam at both ends with displacement at one end reasonably approximates the steel pipe crossing the soil / geologic differential settlement (see Figure 4). Although the length over which this differential settlement can occur can vary reasonable approximations can be made. The spans considered for steel include 10, 20, 30, 40, and 50-foot spans. Most of EBMUD Cast iron pipelines are approximately 10-foot segments. The cast iron pipe was analyzed as a 10-foot span as a cantilever beam with displacement at the free end to represent the cast iron pipe crossing the boundary, experiencing liquefaction induced settlement, as shown in Figure 5.

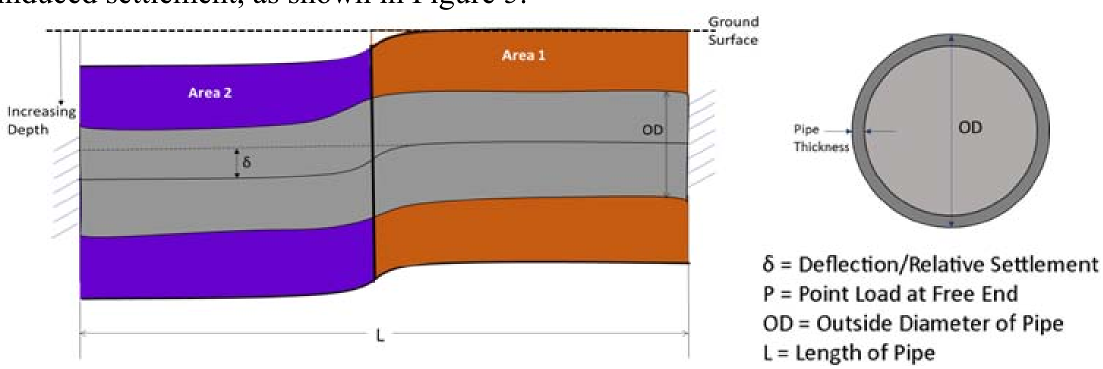

Figure 4: Steel Pipe analyzed with fixed ends and Typical Pipe Cross Section

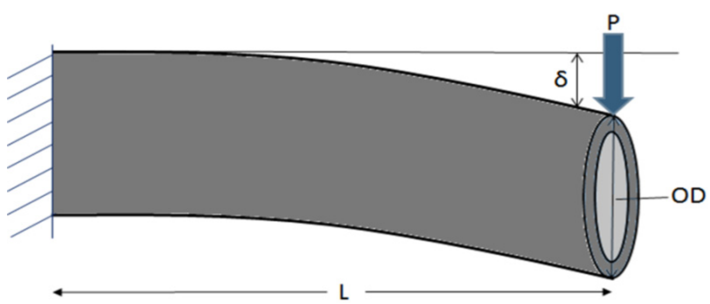

Figure 5: Cast Iron Pipe analyzed as cantilever with point load at free end

Some of the properties for steel and cast iron used for this analysis are shown in Table 2. The elastic modulus and yield stress were used as well as the dimensions of the pipe and range of deflections or displacements acting at one end of pipe to determine the maximum bending moments. Next, these bending moments were computed to calculate the yield bending stresses and strains. The calculations were set up in Excel and verified with Rapid Interactive Structural Analysis (RISA), structural analysis software.

Table 2: Select Properties of Steel and Cast Iron used for Structural Analysis

Material and Pipe Size	Steel, 8"	Cast Iron, 6"
Span (feet)	50	10
E (ksi)	29000	14500
Grade or Yield Bending Stress (ksi)	30	26
Yield Bending Strain = $\frac{\sigma_{yield}}{E}$	0.00103	0.00179
Moment of Inertia (in^4)	32.2	41.5
Section Modulus (in^3)	7.47	12.0
for d = 3 in		
Max. Bending Moment (k-ft)	$\frac{6EI}{L^2}d$ = 3.89	$\frac{3EI}{L^2}d$ = 31.34
Bending Stress (ksi) = $\frac{M}{S}$	6.25	31.27
Bending Strain = $\frac{\sigma}{E}$	0.000216	0.00216
Exceeds Yield Bending Stress?	No	Yes
Exceeds Yield Bending Strain?	No	Yes
References: EBMUD Design Division Standard Drawings, FE Reference Handbook, The Engineering ToolBox, Mueller Company, Standard Handbook for Civil Engineers		

The spans for steel and cast iron pipes were fixed for the calculations. For cast iron, the span was set to be 10 feet. For steel, it is difficult to predict at what distance along the pipe the deflection will occur due to liquefaction induced settlement, so the analysis was run to check 10, 20, 30, 40, and 50-foot spans, as presented in Figure 7.

The steel and cast iron pipes may fail before or after reaching the yield state, but the deflection at yield bending stress was defined to be the point at which the pipes would fail. The deflection and yield stress results were plotted as shown in Figures 6 and 8. The larger diameter pipes can resist more deflections than the smaller diameter pipes. The deflection at yield bending stress for the different pipe sizes was determined using Excel's Solver.

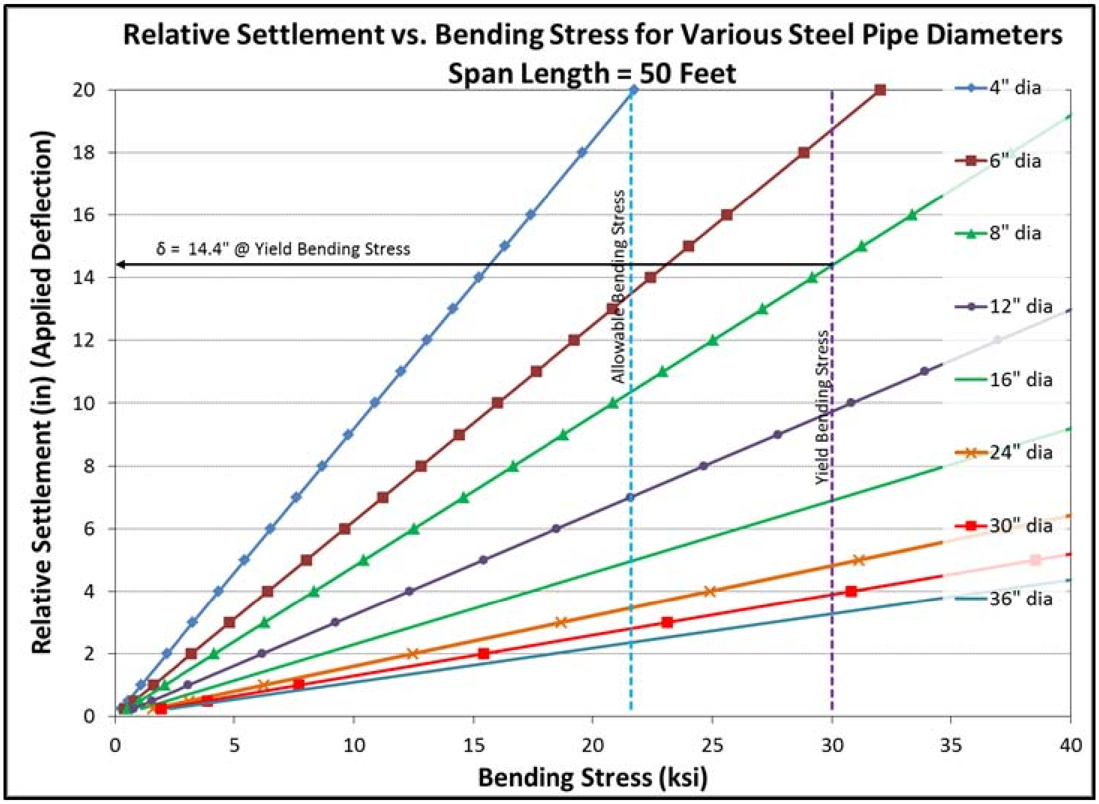

Figure 6: Relative Settlement vs. Bending Stress for Various Steel Pipe Dia.

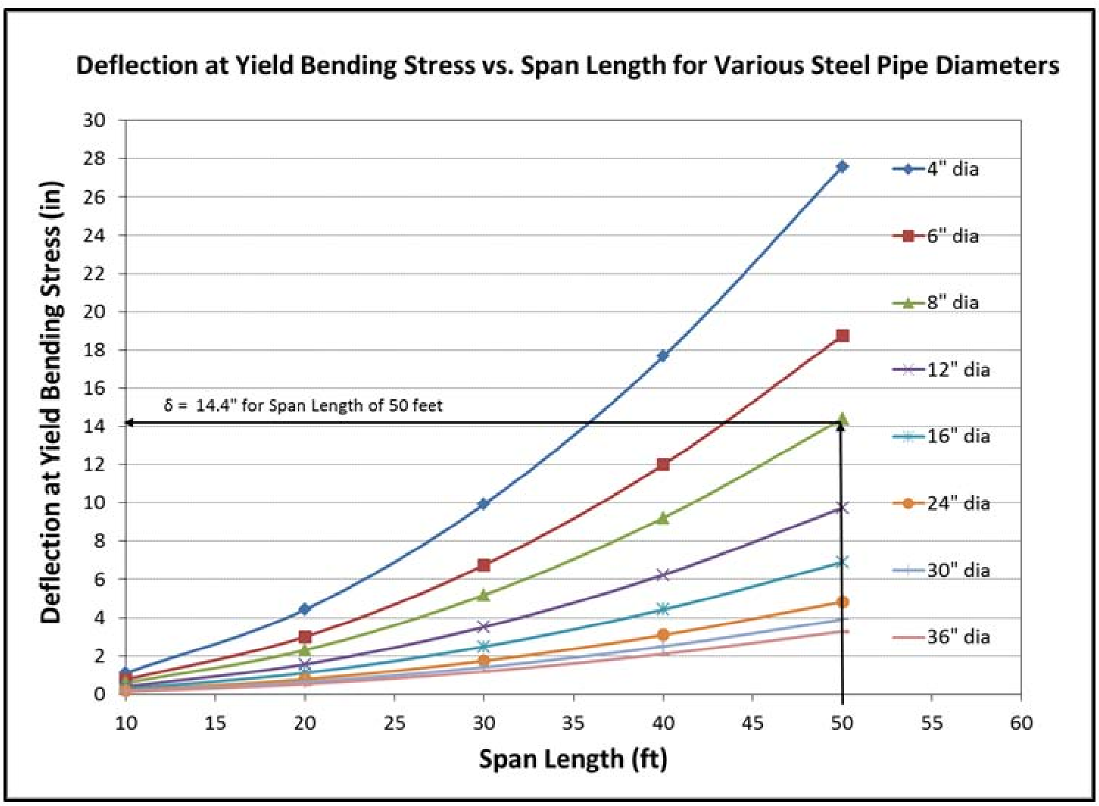

Figure 7: Deflection at Yield Stress vs. Span Length for Various Steel Pipe Dia.

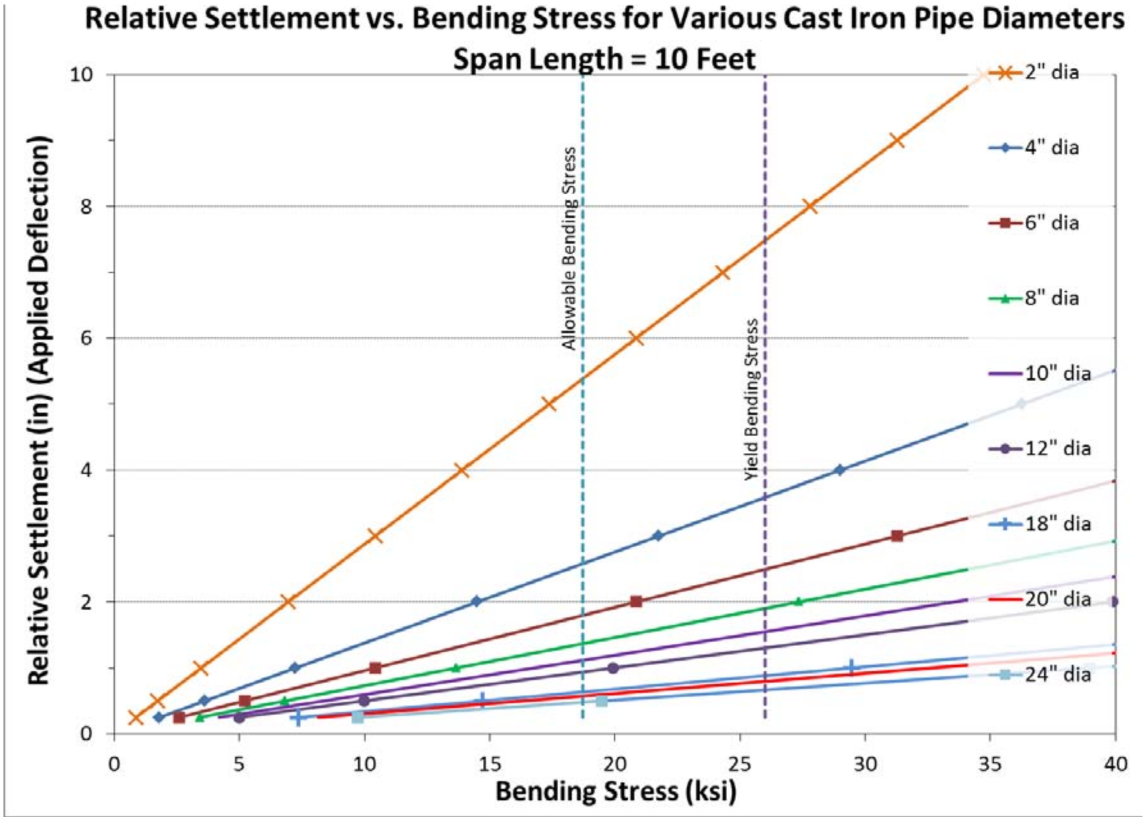

Figure 8: Relative Settlement vs. Bending Stress for Various Cast Iron Pipe Dia.

GIS AND RESULTS

The steel pipes evaluated at the study area boundaries included 8", 12" and 24" nominal pipe size diameters. The cast iron nominal pipe size diameters reviewed at the study area boundaries included 4", 6", 8", 10", and 12" sizes. The deflections at the corresponding yield bending stresses for the different diameter pipes were linked to the Excel spreadsheet containing the relative settlement estimates for Areas 1 and 2, Areas 3 and 4, and Areas 3 and 5. The spreadsheet was set up to flag pipes with deflections at the yield bending stress that exceeded the relative settlements. The district used the relative settlements for a PGA of 0.5 g (high fragility) and compared the results to yield bending stresses of the steel and cast iron pipes. The yield bending stresses for the steel pipes were based on the 50-foot span and 20-foot span.

For GIS purposes, the project team decided to focus on the 20-foot and 50-foot spans for steel pipelines to compare the overall range of relative settlements. GIS was used to determine and show what pipes failed in Areas 1 through 5. More cast iron pipes failed than steel pipes. The pipes crossing Areas 3 and 4 experienced the most pipe failures. Table 3 and Figures 9 through 11 present the damage results for steel and cast iron pipelines.

Table 3: Summary Table of Results of Damaged Pipelines

PGA of 0.5 g (Fragility)	Type	Total Number of Pipes	Number of Pipes Exceeding Yield Stress	% of Pipes Exceeding Yield Stress	Number of Pipes Exceeding Yield Stress		
					Areas 1 & 2	Areas 3 & 4	Areas 3 & 5
Low	Steel	134	0	0.0%	0	0	0
Low	Cast Iron	142	20	14.1%	12	6	2
Average	Steel	134	4	3.0%	4	0	0
Average	Cast Iron	142	131	92.3%	56	64	11
High	Steel	134	4	3.0%	4	0	0
High	Cast Iron	142	142	100.0%	56	75	11

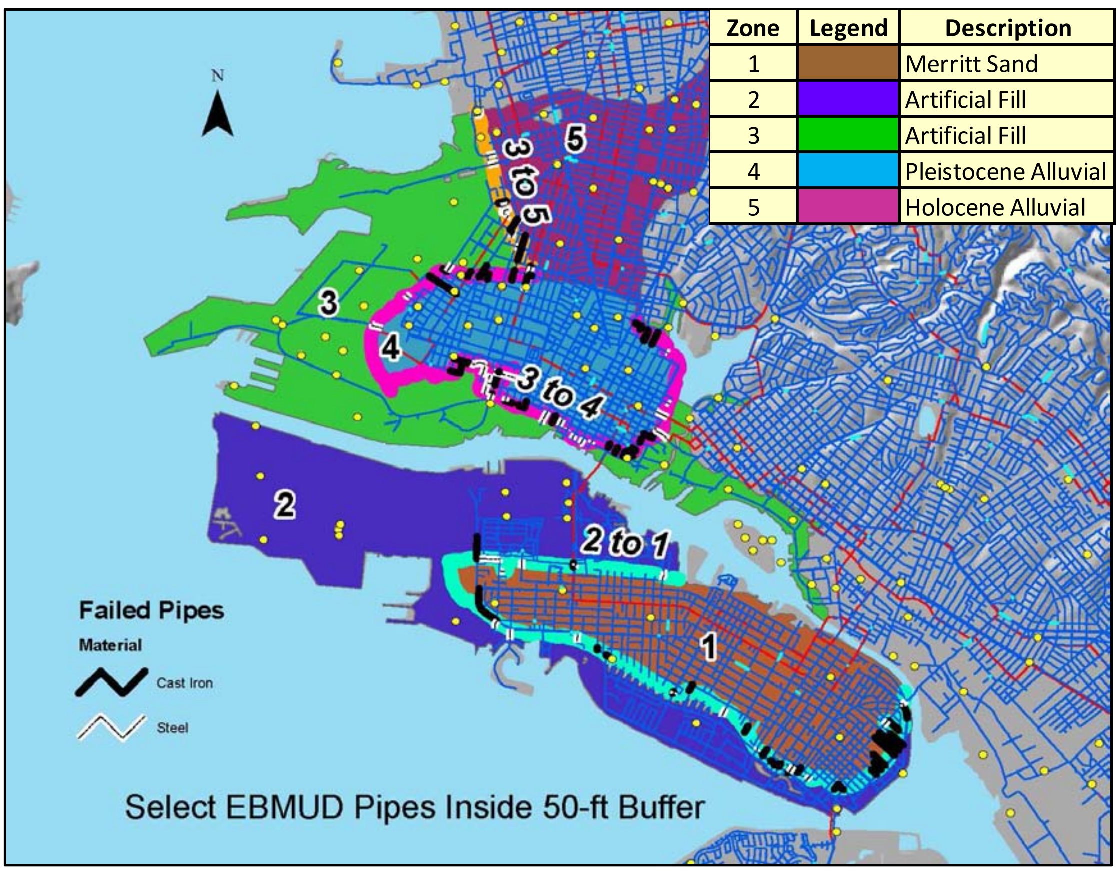

Figure 9: GIS Map of Failed Steel and Cast Iron Pipes

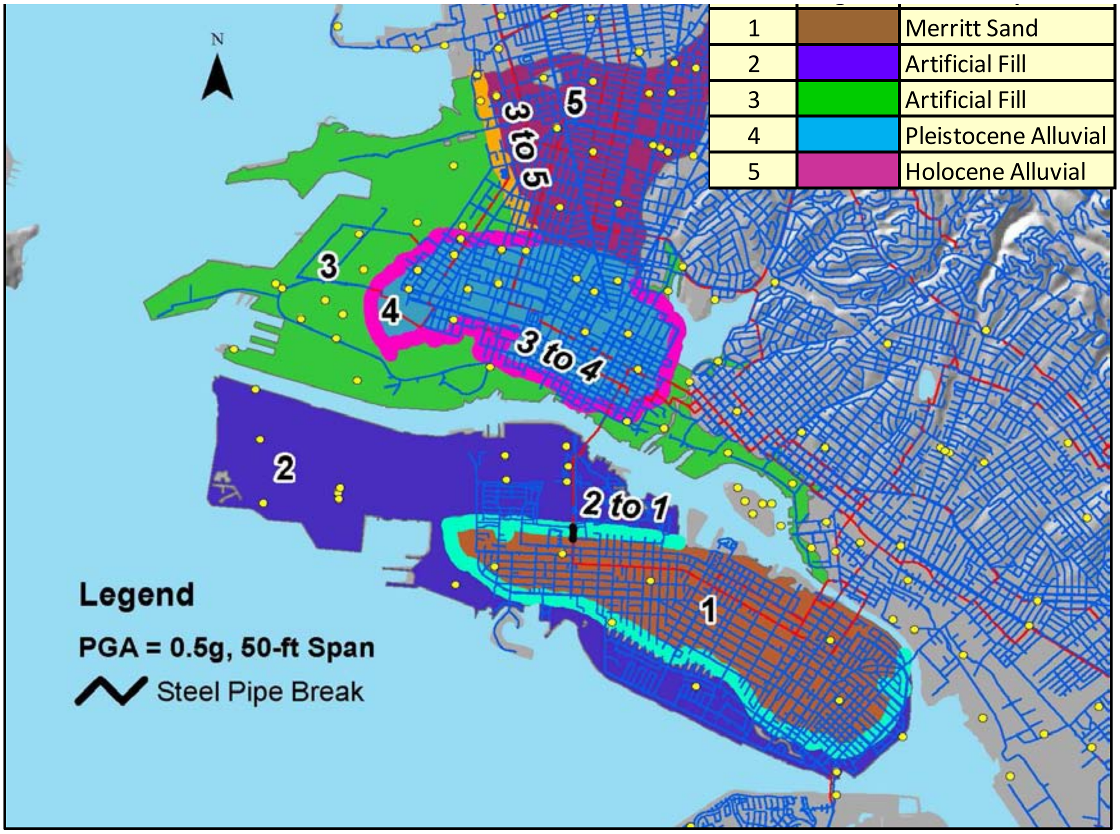

Figure 10: GIS Map of Failed Steel Pipes for 50-ft Span

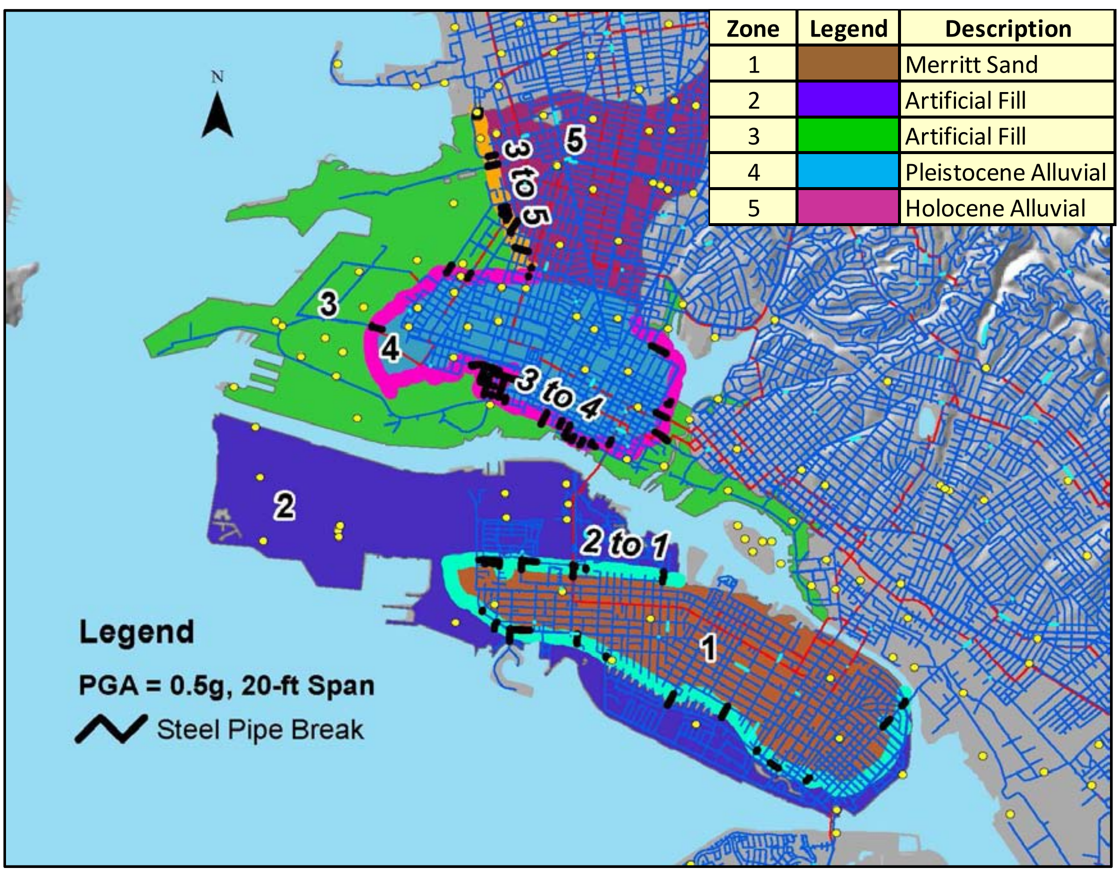

Figure 11: GIS Map of Failed Steel Pipes for 20-ft Span

SUMMARY AND CONCLUSION

Liquefaction induced settlement can cause extensive damage to buried pipelines as recorded from previous earthquakes. These breaks can result in loss of service (including firefighting capabilities) in many areas. The results of this study's evaluations predict many of the steel and most all the cast iron pipes that cross liquefaction boundaries will very likely be damaged at a minimum PGA of 0.5 g using the upper bound estimate for relative settlement. The majority of steel pipes with 20-foot spans failed when compared to steel pipes with 50-foot spans. Choice of pipeline span length is crucial in evaluating the performance and can result in a range of performance results depending on which span length is considered. This study suggests the use for steel pipes with 30-foot spans when considering pipeline performance within a particular zone. For pipelines traversing across different zones (say Zone 1 and Zone 2) we suggest using a shorter span length of 20 feet. Based on the results, the District should replace cast iron pipelines in areas of high relative settlements since they will most likely rupture during an earthquake event.

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Guidance from Tunnel Impact Analyses for DC Clean Rivers Project: Design-Build Bidding to Protect Critical Pipelines

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Abstract

The backbone of the DC Clean Rivers Project includes the construction and operation of 23-foot finished diameter tunnels. The tunnel alignments total almost 15 route miles, and traverse underneath existing sewage and water pipes that are paramount for the residents, federal government offices, and businesses in Washington, DC. These existing pipelines have to serve the rate payers for many decades; therefore, serviceability of these existing pipelines cannot be impacted adversely by the construction and operation of the tunnels. The authors undertook a detailed investigation of anticipated impacts on several pipelines by two of the tunnels. This paper presents the data and methodology used, and the results obtained. Based on the outcome of these analyses, the ground movement monitoring plan was designed and implemented. A summary of how the design-build bidding process benefited from the above risk management-based impact analyses is also presented.

INTRODUCTION

Impacts to buried pipelines due to tunneling ground movement are frequently investigated following the analytical methods described by Attewell et al. (1986), Trautmann and O'Rourke (1982), and others. While the general framework of these methods has applicability to a variety of pipe types and tunneling scenarios, the above primarily address the situation of homogenous pipe types such as cast iron, ductile iron, polyvinyl chloride (PVC), and high-density polyethylene (HDPE). Predicting the impacts of tunneling becomes more complex when considering pipe types such as prestressed concrete cylinder pipe (PCCP) and reinforced concrete cylinder pipe (RCCP) because of their composite construction and the possibility of unique failure mechanisms. In these cases, additional attention must be given to the details of the pipe design, pipe-soil interaction, installation conditions, and operating conditions when considering the impact from tunnel excavation settlements and whether the anticipated settlements are tolerable.

DC Water is undertaking several projects associated with a long-term combined sewer overflow control plan, collectively referred to as DC Clean Rivers (DCCR). The Blue Plains Tunnel (BPT) and the Anacostia River Tunnel (ART) projects are two of the larger tunnel projects that DC Water is executing. Each has water mains and force mains constructed of PCCP or RCCP located within the predicted zone of influence (ZOI) of these construction projects. These mains range in size from 30 inches to 108 inches in diameter, and some are considered fragile because of their history and because they were installed with Class IV wire. Because of the critical nature of these pipelines to current operations and the need to establish safe thresholds for monitoring pipe movements during construction, DC Water undertook a more detailed analysis of the structural impacts than what might be considered standard practice.

The analyses were completed by a team with knowledge in geotechnical engineering; mechanics; PCCP and RCCP standards; AWWA standards C300, 301, and 304; and AWWA *Manual of Practice M9* standards governing the design of PCCP and RCCP. Insights were gained from the following areas of analysis:

- Changes in mortared-joint behavior at increasing levels of pipe movement,
- Distribution of stresses and strains in the pipe components (core, cylinder, and coating),
- Long-term serviceability of the pipes in terms of severity and location of damage to specific joints, ability to inspect the damage, and overall pipe reliability,
- The level of ground volume loss at which the pipes are no longer susceptible to damage from tunnel construction,
- Investigation of the causes of a historic failure of a 108-inch PCCP force main during Washington Metro Area Transport Authority (WMATA) twin tunnel excavation settlements and this failure's applicability to DC Clean River PCCP and RCCP.

The results of the analyses were used to specify the maximum allowable tunneling-induced ground and pipeline movements to be used by the design-build teams in preparing their bids and executing the work.

GENERAL METHODOLOGY

The most common approach to analyzing the impact of tunnel construction on existing pipelines has been to predict the settlement trough resulting from the tunnel's construction and estimate the affected pipe's tensile strain, joint rotation, and joint pullout to determine what protective measures are needed. This basic approach has application to the evaluation of PCCP and RCCP, but it became pressing for the DCCR team to consider the matter in an expanded manner, accounting for factors that may be neglected for less critical and homogenous pipe types.

The behavior of the PCCP and RCCP, which have mortar-filled joints, in response to tunnel settlement depends on the properties of the pipeline and the magnitude and the location of the stresses and strains. Once a pipe joint filling began to crack, it would allow the joint to rotate. The authors calculated the bending moment and the beam shear force using a continuous beam model, following a green field settlement analysis as if this beam is extremely flexible, and then lowering this bending moment to reflect the actual pipe-soil relative stiffness. The authors also calculated the joint rotation and corresponding pullout length at the bell-spigot assuming the pipeline turned into a jointed structure due to the mortar filling in the joints reaching its tensile limits. The stresses, strains, joint rotations, and joint pullouts anticipated in the field would fall within the results from the above two bounds, defined by the behavior of a continuous beam and that of a beam asymptotically approaching a hinged structure. Finally, the authors investigated the effect of compressive stress at the lip of the joints and the possibility of a lip shear failure. The procedure followed is outlined below and described in additional detail in the subsequent sections.

Table 1. Evaluating Tunnel Settlement Impacts on Critical PCCP/RCCP

Evaluation Step	Methodology	Basis for Acceptance
1. Estimation of green field ground movement from tunnel excavation	Mair (2008)	Consider a range of ground movement profiles, varying the volume loss and ZOI width
2. Pipe deflection, bending moments, and beam shear 3. Pipe strain: bending, axial, initial condition, Poisson's effect, temperature, internal pressures	Vorster et al. (2005), AWWA M9, Jeyapalan et al. (1987), and principles of engineering mechanics	Limit tensile strain to prevent onset of visible cracking per AWWA M9 and C304
4. Joint rotation	Using movements at joints and piecewise linear form	Limit to manufacturers' suggested thresholds
5. Joint opening	Using rotation at joints and effects of lateral ground movements	Limit to manufacturers' suggested thresholds
6. Joint lip shear	Principles of engineering mechanics and reinforced concrete design	Limit to factored tensile strength or shear strength of mortar

The biggest challenge for the design team was to decide how to apply the limiting values that are usually interpreted from codes and standards for the installation of new pipelines to existing pipelines in the ground needing to be evaluated.

GREENFIELD GROUND MOVEMENTS

A realistic prediction of the green field movements is crucial for estimating the bending moments, beam shear forces, and the axial forces exerted by the vertical and lateral movements of the ground from tunneling. Most widely used empirical prediction methods of the vertical settlement profile transverse to the tunnel alignment employ a Gaussian distribution form or variation thereof (Cording and Hansmire, 1975).

Tunneling will also cause lateral soil movements that tend to flow toward the tunnel axis. The empirical method of estimating the settlement and lateral movement was selected over numerical, geomechanical modeling so that more control could be maintained over the range of settlement profile shapes considered for the analyses. The primary parameters defining the empirical settlement trough are the assumed ground loss resulting from the tunneling process (expressed as the percentage of the unit volume of the settlement profile in relation to the theoretical tunnel excavation unit volume) and the width of the settlement profile.

Ground loss is related to soil conditions and tunneling methodology. The ART and BPT excavations are approximately 27 feet in diameter and 55 to 100 feet below the ground surface. The tunnels will be excavated using tunnel boring machines, each with a pressurized cutter head chamber, and will be lined with precast concrete segments with gaskets to minimize ground loss. Based on recent case histories under similar conditions, it was determined that the volume of ground loss (VL) from tunneling could reliably be less than 0.5% to 1.0% for the pipelines of interest. Analyses for each pipeline considered ground losses of 0.125%, 0.25%, 0.5%; ground loss of 1.0% was considered for a couple of cases.

The width of the transverse settlement profile has been shown to be related to the soil type, soil consistency, and the depth to the tunnel from the elevation of interest (Mair, 2008). The width of the settlement profile is commonly calculated by determining the distance from the tunnel centerline to the inflection point, i.e., the point at which the curvature of the profile changes from concave up to concave down.

The reliability of green field settlements at the pipe axis predicted using the Gaussian form is highly dependent on the choice the designer makes for the distance of the inflection point, i. The value of i significantly influences the magnitude and location of stress and strain induced in the pipeline. Therefore, a range of settlement profile shapes was estimated for each pipeline with an i-value in the range of 30 to 52 feet. Several settlement profiles considered for the evaluation of a 108-inch PCCP sewer force main are shown in Figure 1.

PIPE BENDING MOMENTS FROM TUNNEL-INDUCED SETTLEMENT

Pipes were initially considered continuous, extremely flexible beams with bending moments calculated using the pipes' curvature, or the second derivative, and beam shear using the third derivative of the settlement curves. The moments were then corrected to account for the relative stiffness between the pipe and soil, as described by Vorster et al. (2005).

The pipe-soil stiffness parameter[1] R is defined as R = EpIp/ (EsRpi^3), where EpIp is the pipe sectional rigidity, Rp is the mean radius of the pipeline, and Es is the Young's modulus of the soil at the average strain in the soil surrounding the pipeline.

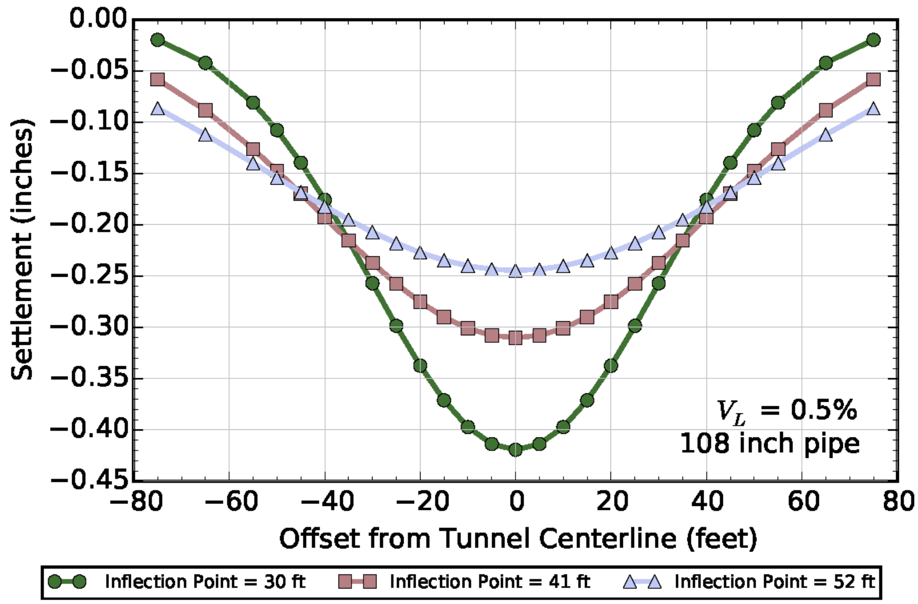

Figure 1. Greenfield settlements with varying inflection point value

The behavior of a pipeline as either a continuous very flexible beam or an asymptotically hinged structure is not solely governed by its own properties or its joint behavior. It is more dependent on the magnitude of the above pipe-soil stiffness parameter, R. For example, a pipe-soil system with an R value close to 0.01 will have moments in the field almost equal to those calculated with the very flexible continuous beam assumption, while an R value close to 5 would result in the very flexible continuous beam assumption giving much higher bending moment values than those anticipated in the field. To account for the effect of pipe-soil stiffness, appropriate correction factors were calculated for each pipeline, as a function of R, and applied to the moments and settlements, which were calculated using the very flexible continuous beam assumption.

Typical results for the three types of bending moments are shown in Figure 2. The moment of inertia of the composite pipe cross section in beam bending mode involved the recognition of the neutral axis shifting above the spring line because of the assumption that only the steel cylinder in the tensile side of the pipe is capable of carrying tension as a result of the concrete core and cement mortar being cracked.

[1]The form of Vorster's pipe-soil stiffness parameter R used for predicting the longitudinal interaction of the pipe and soil mirrors the pipe to soil stiffness factor, PSR = EpIp/0.149 E′ R^3_p, introduced by Jeyapalan and Abdel-Magid (1984) for predicting the circumferential behavior. This PSR has been widely accepted in buried pipeline design as the primary dimensionless group that would determine the behavior of the soil-pipe system and even for the classification of whether a given pipe is flexible, semi-rigid, or rigid.

The corrected bending moments in the pipe were used to estimate the bending strain caused in the outermost fibers in cement mortar, steel cylinder, and the concrete core.

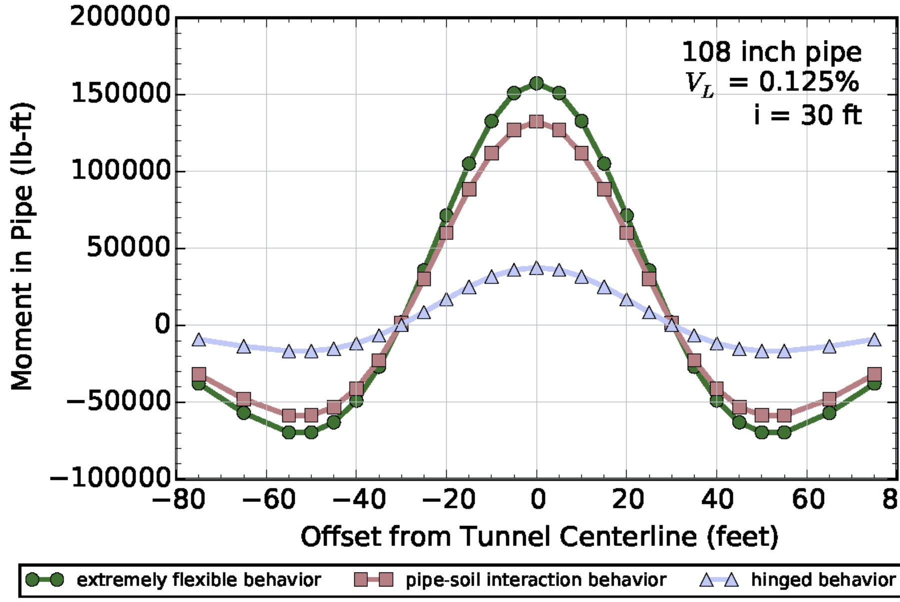

Figure 2. Variation of moments with pipe stiffness and joint stiffness

PIPE STRAIN FROM TUNNEL INDUCED LATERAL MOVEMENT

In addition to bending moments from settlement, lateral ground movement also causes axial stresses and strains in the pipe. Lateral movements cause compressive stresses and strains within the middle portion of the ZOI (where sagging moments from tunnel-induced vertical movements are present) and tensile stresses and strains past the point of inflection (where hogging moments from tunnel induced vertical movements act). The first derivative of the lateral movement profile at the pipe axis was used to estimate the strains caused by the tunnel-induced lateral movement. The bending strains in the pipe over the middle portion within the ZOI are tensile in the bottommost steel, core, and coating fibers at the pipe invert, and compressive at the top of the invert. When these bending strains are combined with the compressive lateral strain, the result is that the tensile strain from bending is negated, assuming that the lateral strain in the soil is transmitted to the pipe at 100% efficiency (i.e., the interface between the outer skin of the pipe and the surrounding bedding material is a perfect bond). In the field, however, we would not see this benefit to such an extent. The compressive axial strains exerted by the lateral soil movement on the pipe in the sagging moment zone would transmit from 0% to at most 30% from the soil to the pipe, based on the skin friction angle for good quality bedding material used during installation being 17 degrees. To be sure that the investigation was broad enough to include all possible cases in the field, 0%, 30%, and 100% transmission of axial strains to the pipe were considered in this study, but in the final evaluation only 30% was used.

The effects of lateral soil movement beyond the inflection point, in the hogging zone of the pipe, however, yield different results. In this area, both strains at the uppermost fibers in steel, core, and coating of the pipe crown are tensile and therefore add to one another to become a controlling serviceability criterion for the pipelines and their joints in some cases. The transmission of the lateral strain from the soil to the pipe in the hogging zone was also assumed to be 30% efficient.

DETERMINATION OF TOTAL PIPE STRAINS

The authors performed calculations for the tunnel-induced tensile and compressive stresses and strains in the steel cylinder, concrete core, and mortar coating due to bending moments, allowing for pipe-soil interaction, and lateral ground movements. To complete a detailed assessment of the risk to the water and force mains, it was necessary to account for the initial conditions, thermal effects, and internal pressures, and thereby permit an estimate of the total strain to which the pipes could ultimately be subjected. An example of the initial conditions considered is that of a 48-inch water main that sloped in and out of the Anacostia River and was supported on piles. These details were captured in the calculations done for the impact assessment. Maintenance and inspection records also need to be examined in such investigations.

Tensile stresses and strains will develop from thermal effects if the pipelines are installed at a temperature higher than the coldest temperature they could ever reach during the pipelines' service life. In the absence of any historical weather or concise pipe installation records that would indicate which pipe length was laid when, a 25°F differential was used to calculate the thermal strain. Poisson's effect–induced tensile stresses and strains were also considered using the total operating pressures or maximum field test pressures.

When it became obvious that the highest compressive stresses calculated in the steel cylinder, the concrete core, and the mortar coating were of no major concern, the focus of this investigation turned to the evaluation of total tensile strains in these materials under all probable loading conditions in the field. The total strains in the outermost fibers of the steel cylinder, the concrete core, and the mortar coating for a typical case are shown in Figure 3.

Initial baseline strains in the pipe arise from two sources: (1) those due to variations from a plane strain assumption where every cross section along the length of the pipe is of almost similar geometry, loading, pipe sectional properties, and other; and (2) those due to variations in the quality of the pipe used and the quality of construction in the field. In the 48-inch pipe, however, only the initial strains based on the varying conditions along the alignment before tunneling began (but not due to the variations in the quality of the pipe and construction) were included in calculating the total strain. This does not imply, however, that because of their variations in quality and construction, the 36-inch pipe, 96-inch pipe, and 108-inch pipe do not also have initial strains; the authors did not have this information available to include as a component in their analyses. In all the pipes, thermal effects cause 182.5 microstrain in the steel cylinder, and 200 micro strain in the concrete core and the mortar coating. Poisson's effects cause from 10 to 75 micro strain in the steel cylinder, the concrete core and the mortar coating. Those strains from variation in soil cover and pipe support conditions add another 25 micro strain in the sloping 48-inch pipe without accounting for any other variations due to native soil, bedding, construction quality, and other factors. The remaining portion of the total strain comes from the bending caused by the vertical settlement and the lateral movement from the ground loss. All these contributions are added together to estimate the total strain. The total strain in the coating cannot exceed the limits shown in Table 2.

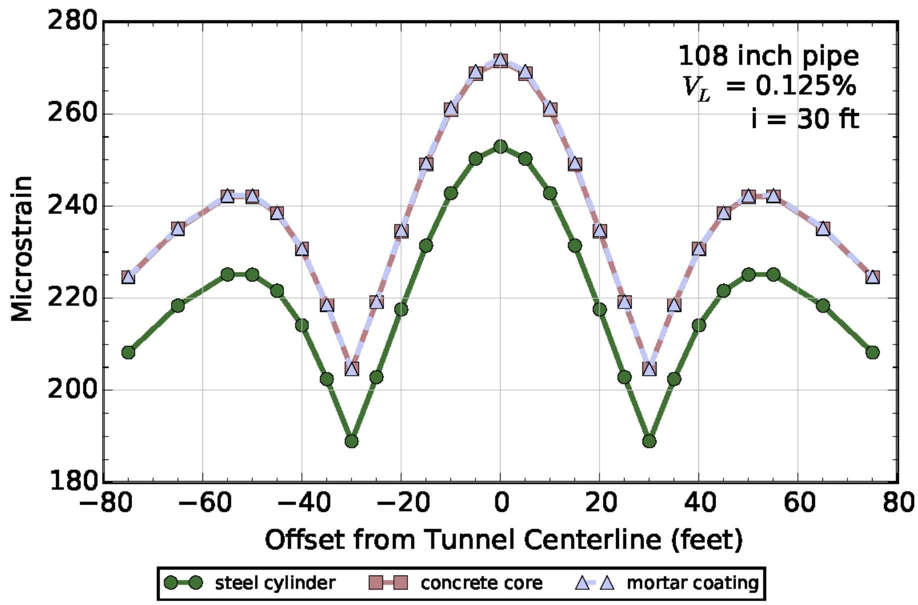

Figure 3. Total strains in steel cylinder, core, and coating

JOINT ROTATION FROM THE HINGED CASE

The joint rotation and the resulting gap were estimated by considering the asymptotic hinge case, in which the pipeline has cracked enough because tensile strains had reached their limit at joint fillings and could no longer assist the pipeline in behaving like a continuous beam with the ability to interact with the soil to resist soil movement resulting from tunneling. The total joint rotation was estimated assuming that the deflection within each pipe length is small, given that prestressed concrete pipe has a very high moment of inertia and therefore is of very high stiffness. These simplifying assumptions allowed the authors to rely on the settlements of the joints or settlements at intermediate points to calculate the maximum joint rotation due to vertical soil movements from tunneling. Representative results for the maximum joint rotations in a 108-inch PCCP force main responding to ART settlements were calculated as 0.06, 0.03, and 0.015 degrees for ground loss levels of 0.5%, 0.25%, and 0.125%, respectively. The tolerable joint rotation for a 96-inch pipe is shown in Table 2.

Table 2. Allowable strain, crack width, joint pullout, rotation and lip shear

AWWA C304-07 Onset of micro cracking (μ)	AWWA C304-07 To control micro cracking (μ)	AWWA C304-07 Onset of visible cracking in coating(μ)	Allowable crack width (mm)[1]	Allowable joint rotation (deg)[2]	Allowable joint pull out for no leakage (mm)[3]	Ultimate shear strength (psi)[4]
184	787	982	2	0.175	19	235

[1]Based on AWWA C300 with some allowance; [2] and [3] are based on the Hanson Pressure Pipe Product and Services Guide and a factor of safety, but primarily governed by the current condition and the original installation of the pipe; the allowable joint rotation ranges from 0.125 to 0.5 degree varying as a function of the pipe diameter; [4] is based on 50% of tensile strength of a coating with a compressive strength of AWWA C300–specified 4,500 psi.

JOINT OPENING

The opening of a joint may occur because of rotation of the joint in response to tunnel settlement or from pullout within the hogging moment area in response to lateral soil movement. The extent to which a joint could open on one side of the pipeline because of rotation was estimated for any joint near the centerline of the tunnel where the maximum sagging moment acts. Fortunately, the tendency for lateral movement in the hogging moment area to pull the joint apart will not take place to a degree close to that resulting from joint rotation in the sagging moment area. Therefore, the worst case scenario would be for the joint pullout length to be only from the vertical-movement-induced joint rotation from tunneling. With the levels of joint pullout predicted in a few cases, the mortar filling the original joint gap of 0.75 inch would crack for certain conditions covered in this study. The permissible joint opening is shown in Table 2.

JOINT LIP SHEAR

Past experience in Washington, DC, when Metro tunnels were constructed by Washington Metro Area Transport Authority (WMATA) in the 1980s, indicates some damage to joints in the 108-inch Anacostia Force Main (AFM), as shown in Figure 4. Therefore, the potential for similar damage because of shearing of the mortar coating from the impacts of ART and BPT was studied in detail. A parametric study demonstrated that the most critical shearing plane in the lip of either the spigot or the bell would initiate from the mortar coating–concrete core interface at the edge of the spigot or bell and would slope at an angle of 45 degrees until it emerges at the outermost fiber in mortar coating, as shown in Figure 5. An examination of the engineering principles showed that the shear plane is always likely to have a tendency to resemble the shape of one-half of an ellipse, with its major and minor diameters governed by the thickness of the mortar coating and the diameter of the pipe. This is because the existing wire, over the outer concrete core, acts like stirrups in shear and does not afford an opportunity for the shear plane to go any deeper in the lip of either the spigot or the bell, past its presence. The component of the total shear force in the direction of the above shear plane, resulting from the total compressive strain from all loadings, and acting over an area in the shape of a segment of the uppermost part of the pipe cross section, would be defined by the length of the chord, the thickness of the mortar coating and the diameter of the pipe. It is important to note that there is no mention of lip shear in AWWA C304, C300, and M9, although the evolving design of the lips with reinforcing mesh by the pipe manufacturers in recent years indicates some progress to solve this problem.

Figure 4. Joint lip shear failure in 108-inch AFM during WMATA tunneling

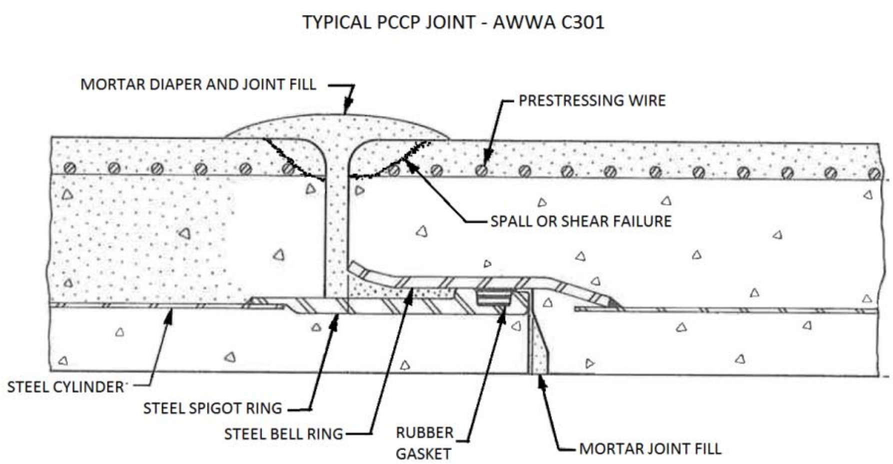

Figure 5. Lip shear failure mechanism from compressive stress at the joint

The magnitude of volume loss experienced on WMATA tunnels was much greater than what is being witnessed on the tunnels excavated in the current project. This is because of the progress our industry has made in its ability to control ground loss. Although lip shear is a concern, it is unlikely a major issue in the DC Clean River Tunnels. Nevertheless, this phenomenon and the mode of failure have not been considered or discussed in any of the national consensus standards, despite historical records revealing that the pipe manufacturers were cognizant of this problem and have tried various ways of improving the design of the lips in the bell and the spigot of the pipe supplied.

SUMMARY

The results from the investigation of the potential impact of ART and BPT tunnel excavations on PCCP and RCCP pipelines—using conservative choices for the value of settlement profile inflection point, i—helped the authors develop conservative bidding specifications for various tunnel contracts. The threshold volume loss levels written into the contracts were to produce a ground settlement no more than ¼ to ½ inch at the pipe level based on the results of the above analysis. Because the existing pipelines were of a critical nature and of a composite construction, and there was a desire to produce as realistic an impact assessment as practicable, the approach and the assumptions needed to be modified from what would commonly be considered for homogenous pipes. Some of the unique considerations of the analyses included:

- Pipe-soil interaction on the basis of their relative flexural stiffness
- Development of flexible and hinged conditions based on changing behavior determined by magnitude of settlement
- Limiting the axial strains in the pipe caused by lateral soil movements on account of the pipe-soil skin friction angle
- Initial conditions, thermal effects, Poisson's effect
- Joint lip shear

Consideration of the total tensile strains, joint rotation, and the potential for joint pullout from

(1) bending due to vertical soil movement,
(2) initial conditions,
(3) thermal effects,
(4) Poisson's effects, and
(5) lateral soil movement

is essential to assure continued serviceability of the pipelines. The analyses undertaken indicate that the risks of angular rotation and joint pullout causing a leakage of the joint are minimal, provided the fabrication and the installation of the pipe joints met the specifications when the pipelines were first installed and no changes of any consequence in their conditions take place over the ensuing decades. There are no concerns about the stresses from beam shear reaching shear strength limits. Also, the maximum compressive strains predicted at the joints and in the pipe barrels are not likely to lead to any crushing of concrete. The compressive strains acting on the lips of the spigot and the bell, however, will have a tendency to shear the coating at the joint by initiating a shear plane at the interface of the coating and the core and completing this crack at the outermost fiber in the coating. This damage to the coating may expose the prestressing wire to moisture, accelerating its corrosion. Cracking of the mortar coating - wider than the 1.5 mm allowed without needing any repair per AWWA C300-11 and C301-07 - is likely in some cases.

To manage risk better, the volume loss and ground movements allowed in the contract specifications were carefully chosen. By setting a lower permissible volume loss, the level of cracking in the joint filling can be minimized needing no preconstruction mitigation measures or strengthening such as consolidation grouting or reinforcing the pipe joints. The findings are heavily dependent upon the values chosen for i. Settlement monitoring to date along the BPT alignment for the completed 3.5 miles shows that settlement profiles assumed for the analyses were reasonably conservative. Settlement along the ART will also be monitored once excavation commences.

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Seismic Fragility Functions for Sewerage Pipelines

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Abstract

Fragility functions (or fragility curves when presented graphically) are a well established tool to assess seismic risk to infrastructures. Given a sustained level of ground motion intensity, seismic fragility functions of sewerage pipelines are used for predicting earthquake-induced physical damage to sewer pipes. However, existing fragility functions of sewerage pipelines refer to a limited number of pipe types and material categories. Therefore practitioners often draw upon fragility functions specifically defined for potable-water pressure pipelines available in the international literature, for foreseeing earthquake-induced failures on sewerage gravity and pressure pipelines. The discrepancies between existing fragility curves and the observed physical damage data collected after the Canterbury (NZ) Earthquake sequence in 2010-2011, evidence that the fragility functions defined for potable-water pipelines tend to underestimate the physical damage to sewerage gravity pipelines. This paper proposes fragility functions for sewer gravity and pressure pipelines, categorized by pipe materials and liquefaction zones. The proposed fragility functions are developed using maximum likelihood estimation by correlating peak ground velocity with damage ratio (defined as number of faults per km) for sewer gravity pipes and with repair rate, defined as number of repairs per km for sewerage pressurized pipelines.

INTRODUCTION

A sewerage system is an indispensable component of municipal infrastructures. The majority of sewerage pipelines are buried deep (up to 15 m) under the ground for making better use of gravitational force to transfer sewage. This exposes the systems to high likelihood of sustaining physical damage caused by seismic ground movements. In particular, sewerage pipelines, being designed only to resist soil loading, are more vulnerable compared to pressurized pipelines when subject to seismic events. Therefore, sewerage systems are significantly susceptible to earthquake hazards.

Buried sewer pipelines experience seismic-induced physical damage through two main sources: wave propagation and permanent ground deformation (PGD). The traveling earthquake waves cause strong ground shaking leading to joint failures and pipe rupture, whereas permanent ground deformation, produced by irreversible ground surface settlement, landslide, liquefaction and associated lateral spreading, could permanently buckle sewer pipes, thus compromising the system functionality. Transient ground motion intends to damage buried pipelines at a system level while PGD preferably cause localized physical defects in a small area (O'Rourke & Liu, 1999).

Fragility functions, graphically illustrated as fragility curves, are a well-established tool to assess seismic risk to buried pipelines, including water supply pipelines (ALA, 2001; FEMA, 2001; Alexoudi et al., 2010; Eidinger, 1998; O'Rourke and Ayala, 1993, 2014; Toprak & Taskin, 2007) and sewerage pipelines (ALA, 2004; Alexoudi et al., 2010; FEMA, 2001; Nagata et al., 2011; O'Rourke et al., 2014). Fragility functions correlate physical representations of ground motion intensity and system damage/repair, represented as numerical incidents per pipe length (Toprak & Taskin, 2007) expressed either as Repair Ratio (RR) (ALA, 2004; Alexoudi et al., 2010; FEMA, 2001; O'Rourke et al., 2014) or as Damage Rate (DR) (Nagata et al., 2011; Shoji et al., 2011).

Fragility curves are used within different platforms for seismic risk assessment e.g. Multi-hazard Loss Estimation Methodology (HAZUS; FEMA, 2001) and Syner-G (Alexoudi et al., 2010), to assess the earthquake-induced damage to pipes. In particular, Syner-G implements ALA (2001), backbone vulnerability curves for water distribution networks in a variation of pipe materials and joint types. HAZUS (FEMA, 2001) uses fragility functions proposed by O'Rourke & Ayala (1993) for brittle and ductile water pressurized pipes for post-earthquake loss estimation. Alexoudi et al. (2010) validates the fragility algorithms available in Syner-G, also proposing fragility functions developed by O'Rourke & Ayala (1993) for water supply pipelines when subject to wave propagation and those proposed by (Honegger & Eguchi, 1992) when water pipelines experience permanent ground deformation. The ALA, HAZUS and Syner-G approaches recommend extending the use of the fragility algorithms specifically derived for water-supply pipelines, when subjected to peak ground velocity (PGV) and PGD, to sewerage pipelines. However, recent seismic events, including 1994 Northridge (America) earthquake (Schiff, 1995), 2004 Niigata (Japan) Earthquake (Scawthorn et al., 2006) and 2010-2011 Canterbury earthquake sequence (Giovinazzi et al., 2011) highlighted that pressurized pipes and sewerage unpressurised (gravity) pipes behave differently under seismic loadings.

A limited number of fragility functions have been specifically defined for sewerage pipelines. Nagata et al. (2011) develops fragility curves for polyvinyl chloride (PVC) sewer pipes within liquefaction and non-liquefaction sites and at various burial depths based on the physical damage data on four cities in Japan after four earthquakes. Shoji et al. (2011) derives a set of fragility curves for sewerage pipelines based on the damage data on the Kobe wastewater system after the 1995 Kobe (Japan) earthquake. They utilize a trust region method for obtaining regression coefficients of the assumed log-normal distribution of fragility curves. O'Rourke et al. (2014) correlates lateral ground strain with RR for earthenware (EW), PVC and unplasticized polyvinyl chloride (UPVC) wastewater pipes; and proposes correlation between the angular distortion, expressed as the differential vertical movement between two adjacent LiDAR points over the horizontal distance, and RR for EW, reinforced concrete rubber ring (RCRR), and concrete (CONC) wastewater pipelines. These fragility functions were generated by processing the data on the earthquake induced damage to the Christchurch sewerage system after the Canterbury earthquakes in 2010-2011.

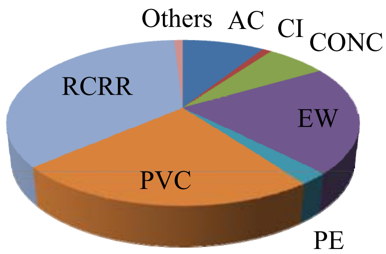
However, there are different shortcomings affecting fragility curves specific for sewerage pipes. The sewerage fragility functions, either only refer to a limited number of pipe types and material categories, or adopt infrequent parameters (e.g. angular distortion) which require extra analytical calculations for practitioners worldwide to obtain and thus leading to added time and resource in assessing seismic performance of sewerage pipelines. Furthermore they use RR as parameters to estimate earthquake-induced incidents to sewer pipes, while DR would provide more reliable and accurate assessment as found by Liu et al., (in review) when comparing and analyzing the databases of the damaged pipes and repaired pipes in the Christchurch sewerage network. Finally, levels of liquefaction extents are not considered in the process of developing fragility functions for sewer pipelines.

In this paper, we assume a log-normal cumulative distribution as the shape of fragility curves of sewer pipelines and propose three-parameter fragility functions by conducting maximum likelihood estimation method. DR of sewer pipelines classified by pipe diameters is calculated for deriving fragility functions for gravity pipelines in five liquefaction zones and one non-liquefaction zone. RR plays a supplementary role when damaged pressure pipe data was not available. The proposed fragility curves of pressure pipelines are compared with those defined for water pressurized pipes in the international literature.

BACKGROUND

The Christchurch sewerage system. Christchurch, the largest city in South Island of New Zealand, has a population of nearly 348,400 living in around 1426 m^2 area. The Christchurch sewerage network with a length of 1835 km, in conjunction with thousands of private laterals, serves 99.9% of the population in Christchurch region. The system is composed of trunk sewer network (pipe diameters ≥300 mm) and minor collector reticulation (pipe diameters < 300 mm). There were more than 42,100 pipes, 25,900 manholes, 164 pump stations, 239 pumps and two treatment plants functioning in the Christchurch wastewater system before the Canterbury earthquakes (CCC, 2011). The Christchurch sewerage system had a variety of pipe materials, as shown in Figure 1 (1). RCRR jointed pipes were the most common pipe type in the Christchurch sewerage network. PVC pipes with welded joints accounted for a large proportion, followed by EW pipes with sealed joints by elastomeric rings. Asbestos Cement (AC) and Cast Iron (CI) pipelines are often jointed by lead.

Although the Christchurch sewerage system is a gravity-fed sewer network, due to the flat Canterbury plains where the system is located, pump stations and pressure pipelines are moderately present in the system. Pressure assets allow for greater sewage



(1) (2)

Figure 1. (1) The proportion of the Christchurch sewerage system classified by pipe materials; (2) physical damage to AC sewer pipes caused by the earthquakes.

flow rates and better prevention from pipe blockage than gravity pipelines. Dated before the earthquakes, there were in total 1681.81 km gravity pipes, which accounted for 91.65 % of the whole sewerage network and 153.23 km long of pressure sewer pipelines in Christchurch. Until now, as part of the post-earthquake rebuilding operations, the proportion of pressure pipelines has increased up to 12.58% of the entire Christchurch sewerage network (SCIRT, 2014).

Seismic performance of the Christchurch sewerage system following the Canterbury earthquake sequence in 2010-2011. Christchurch resides on the Canterbury Plains, a fan deposit formed by numerous rivers flowing eastward from the foothills of the Southern Alps. Soils underneath Christchurch are comprised of a complex sequence of gravels inter-bedded with silt, clay, peat, and shelly sands (Forsyth et al., 2008). According to the soil classification proposed by the National Earthquake Hazards Reduction Program (FEMA, 1997), Christchurch soils can be classified as class E, "soft soil". The main surface layers in the west and east of Christchurch are the Springston formation (containing alluvial gravels, sands and silts) and the Christchurch formation (estuarine, lagoon, beach, dune, and coastal swamp deposits of sand, silt, clay and peat) (Forsyth et al., 2008). The spatial variability of foundation soils in the near-surface geology of Christchurch can explain the unevenly distributed liquefaction and lateral spreading occurred in the west and east parts of Christchurch during the earthquakes (Cubrinovski et al., 2011a).

Among the Canterbury earthquake sequence that struck Christchurch (three most destructive ones: Mw 7.1 September 4 2010, Mw 6.2 February 22 2011 and Mw 6.3 June 13 2011), the February quake is well-recognized as the most severe one, costing 186 fatalities and triggering widespread damage to buildings and city infrastructures, especially in the Central Business District (CBD). Within the CBD area, PGA values ranged from 0.37g to 0.52g (Cubrinovski et al., 2011a).

As a result of the Canterbury Earthquakes, the Christchurch wastewater system experienced extensive damage. 659 km (41 %) of the total sewer pipeline and 136 sewer pump stations throughout Christchurch suffered physical damage at various extents (SCIRT, 2014, Figure 1 (2)). A large amount of fracturing and collapse of brittle pipelines, especially in EW and RCRR pipelines, was observed. Compression failures caused by strong ground shaking and/or land movement occurred mostly on pipe joints. PVC and PE pipes performed reasonably well. Figure 2 presents the number of faults on sewer pipelines classified by pipe diameters and materials.

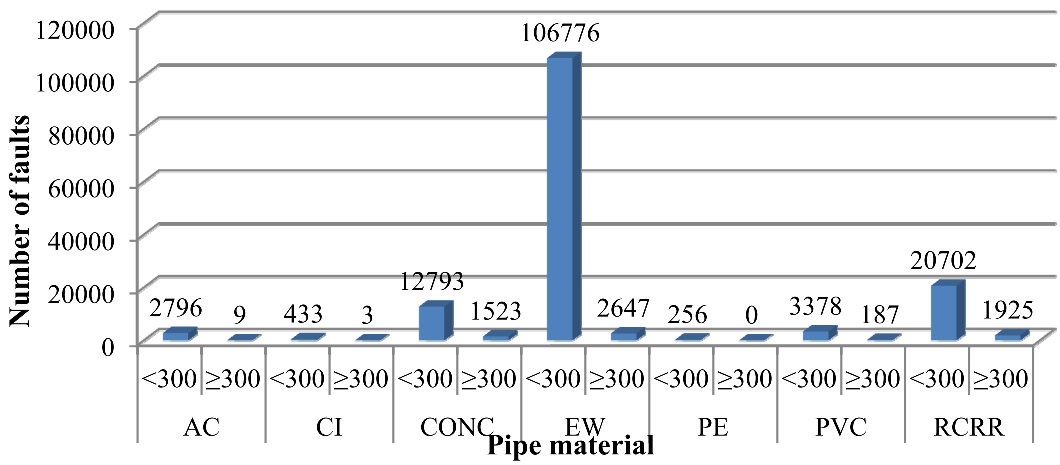


Figure 2. Number of faults on sewer pipelines classified by pipe materials and pipe diameters.

After the Canterbury earthquakes**,** a holistic investigation aiming to assess earthquake induced damage to the Christchurch sewerage system commenced, primarily by means of Closed Circuit Television inspections, in conjunction with other survey methods, such as manhole level survey and pipe profilometer (Liu et al., 2013). After physical defects in sewer pipes were recorded by CCTV surveys, trained assessors embarked on a coding program in accordance with New Zealand Pipe Inspection Manual (NZPIM; NZWWA, 2006). The NZPIM provides standard technical specifications for carrying out CCTV inspections when structural condition of wastewater pipes is required, both during normal operation and after a disaster. It regulates good practice procedures for implementing CCTV inspections in New Zealand and provides a standardized set of codes for processing and analyzing the observational information.

DEVELOPING FRAGILITY FUNCTIONS FOR SEWERAGE SYSTEM PIPELINES

Data collection and processing. Three databases analyzed herein include: 1) sewer pipeline inventory; 2) CCTV inspection database; and 3) repaired pipe database. These databases are collected and maintained by a local earthquake recovery authority - the Stronger Christchurch Infrastructure Rebuild Team (SCIRT) responsible for post earthquake rebuilding of horizontal infrastructures in Christchurch. The sewer pipeline inventory presents a total of 39177 records in the entire sewer network before the February earthquake. After data screening for removal of "out of service" pipes and other pipe types (e.g. overflow), 34158 records remained in the sewer pipeline inventory database, representing in-service gravity, and pressure pipes installed before February 22 2011. The CCTV inspection and repaired pipe databases are dated January of 2015. By then, CCTV crew had confirmed 13,784 pipes as 'damaged' with at least one fault on individual pipes. Therefore, in the CCTV inspection database, there are 13,784 inspected pipes with various defects. The repaired pipe database documents 6,179 pipes that have been repaired by contractors of SCIRT or CCC maintenance teams. These databases are linked by their pipe IDs. Due to the limited number of repaired pressure pipelines in the Christchurch sewerage systems, we develop one fragility function for sewer pressure pipelines (SPP) instead of a set of functions classified by pipe materials as we did for sewer gravity pipelines (SGP).

In this work, the February earthquake is considered and PGV values of this quake are used to correlate with DR/RR for developing fragility functions. These PGV values were obtained from U.S. Geological Survey (USGS) website. They were recorded from around 50 strong motion stations in the Christchurch and Lyttelton area.

The Liquefaction Resistance Index map (LRI; Cubrinovski et al., 2011b) is utilized herein to partition the physical damage and repair operations occurred in each liquefaction zone. The map was produced by use of extensive field mapping conducted by professional geotechnical engineers after the February Earthquake. The average lateral displacement and ground settlement estimates from the map were combined using vector addition to create a PGD value for each region of the map. Table 1 shows the geotechnical details of the LRI. The categories of six liquefaction zones are adopted in this paper to generate fragility functions of sewerage pipes. The no observed liquefaction zone is labeled as No Liquefaction Observation (NLO) herein.

Table 1. Geotechnical details of the LRI map (Cubrinovski et al., 2011b).

Zone	Equivalent CRR (at water table)	Representative LRI (at water table)	Estimated Ground Settlement (mm)	Estimated Lateral Displacement (relative; transient) (mm)	Equivalent ground strains & thickness of liquefied layer
0	< 0.065	-	> 500	> 400	ε_v > 5%, γ > 4%, H_L = 5 -10 m
1	0.065 - 0.11	0.065	250 - 500	200 - 400	ε_v = 5%, γ = 4%, H_L = 5 -10 m
2	0.11 - 0.16	0.13	50 - 250	40 - 200	ε_v = 3%, γ = 2%, H_L = 4 - 8 m
3	0.16 - 0.23	0.195	20 - 50	20 - 40	ε_v = 1%, γ = 1%, H_L = 2 - 4 m
4	> 0.23	0.26	< 20	< 20	γ < 0.5%, H_L = 0 m
No Liquefaction Observations					

Layers of the PGV map of the February quake, the LRI map and the abovementioned three databases regarding the Christchurch sewerage network were jointly superimposed in Graphical Information System in order to assign PGV values to individual pipes and then formulate fragility functions in various liquefaction zones. Both DR (x) and RR (x) are represented as a function of seismic intensity which is PGV in this case. We assume that the fragility functions follow a log-normal cumulative distribution in a form of three-parameter functions as Equation 1 (Nagata et al., 2011).

$$R(PGV = x) = C\Phi\left(\frac{\ln x - \lambda}{\zeta}\right) \tag{1}$$

where R (x) can be either DR (x) or RR (x), expressing estimated DR/RR values of sewer pipelines given a ground motion of PGV = x, $\Phi()$ is the standard normal cumulative distribution function, C, λ and ζ are function parameters to be estimated with the method of maximum likelihood estimation. In this method, PGV values for each ground motion are assumed independent and the likelihood of the entire data set is the product of the individual likelihoods (Baker, 2014). The fragility function parameters are obtained by maximizing the likelihood equation as Equation 2 in the Microsoft Excel Solver.

$$Likelihood\ (x_i; C, \lambda, \zeta) = C^m \prod_{i=1}^{m} \phi\left(\frac{\ln x_i - \lambda}{\zeta}\right) \tag{2}$$

where ∏ denotes a product over i values from 1 to m, n is the total number of ground motions.

Fragility curves of gravity pipelines. Fragility functions as a function of PGV and liquefaction zones are developed by use of maximum likelihood estimation for six main pipe materials of gravity pipelines, namely: AC, CI, CONC, EW, RCRR and PVC & PE. PVC and PE pipelines were combined together as they are all ductile material and PE pipes performed relatively well during earthquakes with little physical damage. Table 2 shows the calculated function parameters for AC gravity pipes.

Table 2. Parameters of fragility functions of AC sewer gravity pipes.

Liquefaction Zone	λ	ζ	C	Constraint: ζ ≥
LRI-0	51.31	6.76	50.64	6.76
LRI-1	49.79	8.24	80.42	8.24
LRI-2	50.42	10.91	73.01	10.91
LRI-3	53.87	12.44	66.24	12.44
LRI-4	57.43	13.57	59.41	13.57
NLO	45.65	11.96	68.47	11.96

The CCTV inspection has not been extensively conducted in LRI-0 zone and thus there are not many detected faults in this area. This is the reason that the pipes in LRI-0 zones do not possess the highest DR. This also explains why the proposed fragility curves of

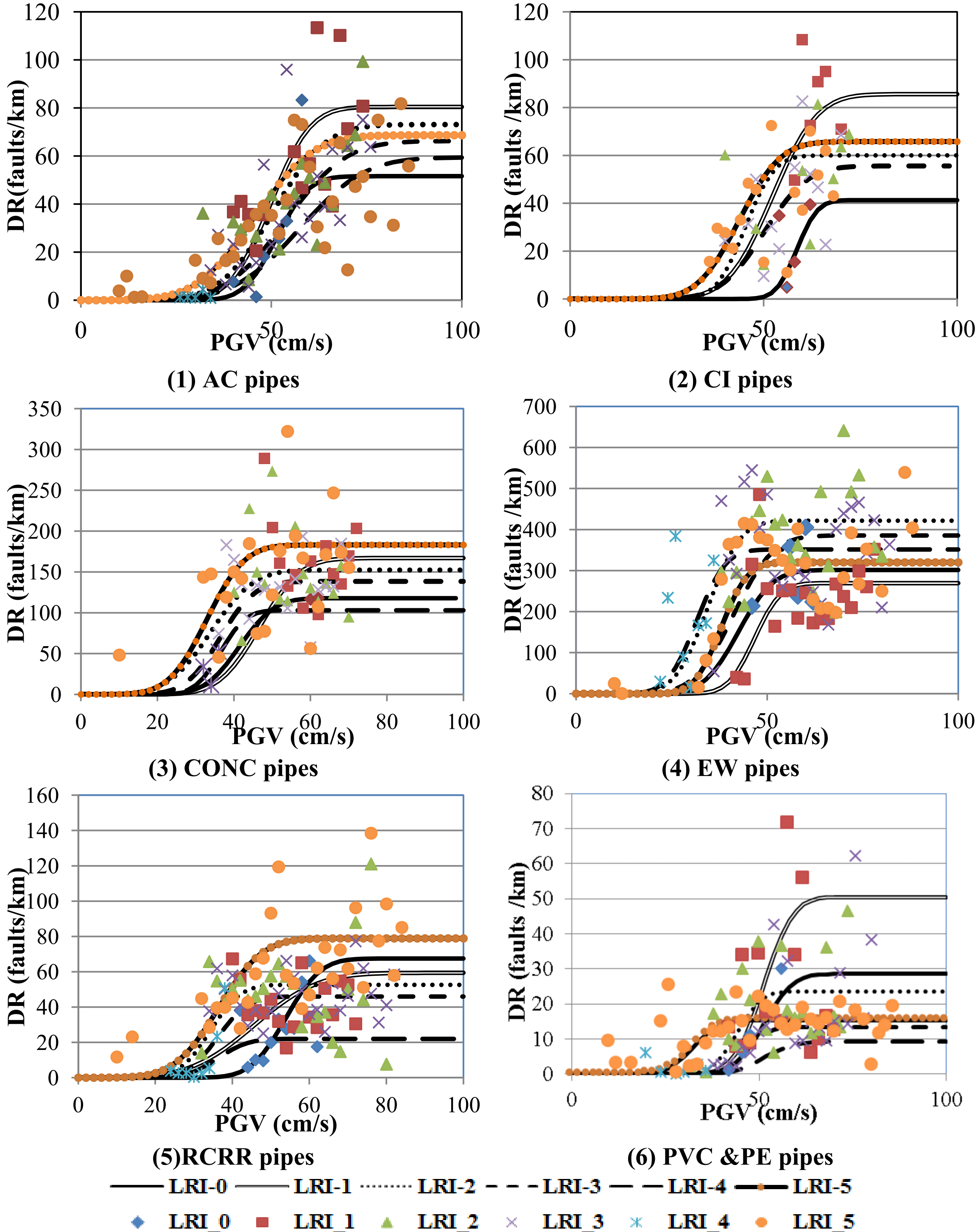


Figure 3. Fragility curves and observed damage data for six types of gravity pipes (namely: AC, CI, CONC, EW, RCRR, PVC&PE) as a function of the PGV and of liquefaction zones.

pipes in LRI-1 zone seem casually distributed. As for LRI-2 and LRI-3 zones, the fragility curves of all types of pipes show an agreement, as expected, that the severer the observed liquefaction is, the higher DR of pipes is found. No faults on CI pipelines were found in LRI-4 zone. In NLO zone where transient ground motion is considered as the only factor, a large amount of physical damage was still observed. CONC and RCRR pipes have the highest damage ratios in NLO zone. It is concluded that damage ratios of sewer pipes in non-liquefaction zones are not necessarily lower than those of liquefaction zones.

Fragility curves of and the observed damage to six types of gravity pipelines, namely: AC, CI, CONC, EW, RCRR, and PVC &PE as a function of PGV in five liquefaction zones and one non-liquefaction zone are plotted and compared in Figure 3. AC pipes and CI pipes behaved similarly during this earthquake event. EW pipes suffered the most severe physical damage to their pipe bodies and the peak damage ratio was found over 400 faults per kilometer in LRI-2 zone. Following EW pipes, CONC pipelines sustained serious earthquake-induced damage, with the damage ratio ranging from 100 faults to 200 faults per km. Although extensive incidents occurred to RCRR pipes, due to the large distribution, their damage ratios are below 100 faults in every kilometer. PVC & PE pipes have the fewest damage ratios among all tested pipes.
In Figure 3, fragility curves of six pipe materials derived for LRI-0 have medium damage ratios and, for AC and CI pipes, have the lowest ones. LRI-0 zone is mostly located in the CBD where a Cordon has been established in view of community safety.

Fragility functions of pressure pipelines. As CCTV inspections have been only conducted on gravity sewer pipelines, there are no recorded faults on SPP. Additionally, the damage to SPP is not systematically investigated and documented by other methods. Therefore, we used repair database which contains repair operations undertaken to SPP to develop fragility functions for them. The fragility functions and associated parameters were generated by the method described above. There were not repair activities undertaken to pressure pipes in LRI-4 zone. Function parameters and fragility curves of SPP are listed in Table 3 and plotted in Figure 4.

Table 3. Parameters of fragility functions of sewer pressure pipes.

Pressure pipes	λ	ζ	C	Constraints: $\zeta \geq$
LRI-0	50.91	8.73	90.53	8.73
LRI-1	53.29	10.12	81.23	10.12
LRI-2	57.59	7.82	18.61	7.82
LRI-3	48.92	10.92	14.27	10.92
NLO	45.56	10.06	21.93	10.06

SPP appear to be quite robust in the low PGV range. They start to sustain repair operations when PGV values increase up to 40 cm/s and repairs rise largely afterwards. Unlike SGP, SPP in LRI-0 and LRI-1 zones sustained a large number of repair undertakings because their functions as connections between pump stations are desirable for the whole sewerage system to operate, especially in LRI-0 and LRI-1 zones. Severe ground settlement (> 250 mm) had significant effects on seismic performance of sewer pressure pipelines compared to median ground settlement (20-250 mm). SPP preformed relatively well when solely subject to transient ground motions, with a repair ratio of around 20 repairs per km. In conclusion, SPP are more vulnerable to permanent ground settlement.

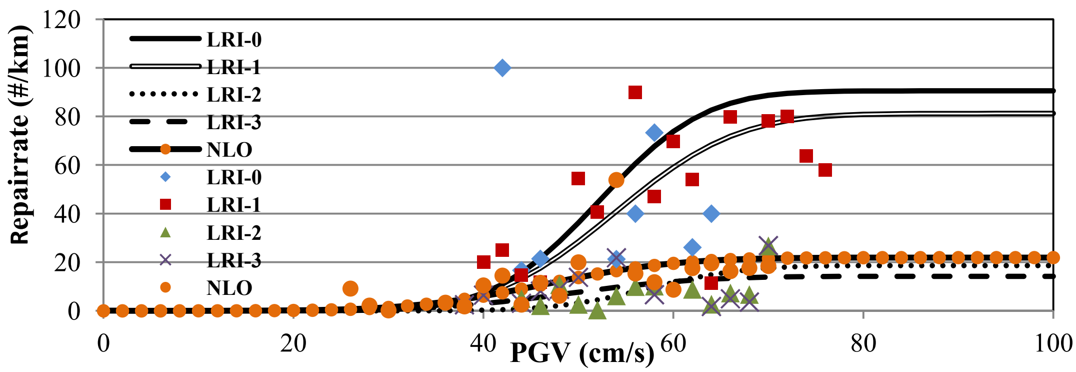


Figure 4. Fragility curves and repairs undertaken for sewer pressure pipelines as a function of PGV and of liquefaction zones.

COMPARING THE FRAGILITY FUNCTIONS DERIVED FOR SEWERAGE PRESSURE PIPELINES WITH WATER-SUPPLY PIPELINE ONES

In order to examine whether the fragility functions of water-supply pressure pipelines (WPP) can be applied to estimate seismic physical damage to SPP, the proposed fragility functions specifically developed for SPP in this analysis is compared with existing fragility curves of WPP in the literature (see Figure 5). AC sewer pressure pipes were selected for comparison purpose herein. We chose k=1 as function coefficient to calculate fragility curve of ALA (2001). HAZUS (FEMA, 2001) and Syner-G (Alexoudi et al., 2010) recommend empirical fragility functions developed by O' Rourke and Ayala (1993) which was illustrated in Figure 5. The fragility function

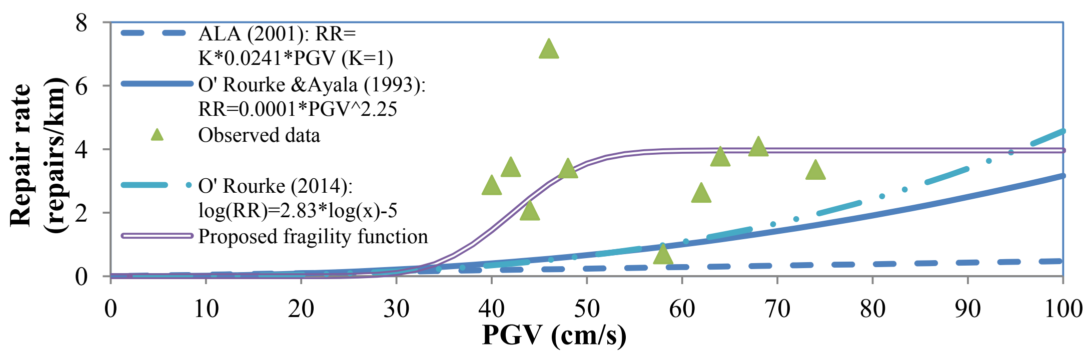


Figure 5. Comparison of the proposed fragility functions of AC sewer pressure pipelines and existing fragility algorithms of AC water-supply pipelines.

developed for AC water-supply pipelines derived from the observed damage data on the Christchurch water supply systems following the Canterbury earthquakes was utilized as well (O'Rourke et al., 2014).

It is shown that there is a certain level of agreement between the proposed fragility curve of SPP and existing fragility algorithms designed for WPP. The existing fragility algorithms of WPP in the international literature slightly underestimated the repairs undertaken on the AC sewer pressure pipelines in Christchurch. After the February earthquake, severe liquefaction and associated lateral spreading occurred near waterways where a number of SPP were installed. This leads to more repair undertakings of SPP to happen for regaining the sanitary service. Furthermore, the uncertainties generated due to different sources of PGV values and various soil conditions surrounding sewer pipelines are of relative influence on the comparison results.

CONCLUSION

This study developed fragility functions for gravity and pressure sewer pipelines by analyzing digital data on the Christchurch sewerage pipelines. The PGV values of the February 22, 2011 earthquake were deployed as a representative of seismic ground motions to correlate with DRs of SGP and RRs of SPP, respectively, in five liquefaction zones and one non-liquefaction zone. The proposed fragility functions of sewerage system pipelines were assumed as a shape of log-normal cumulative distribution. The parameters in the fragility functions were calculated by conducting maximum likelihood estimation method in the Microsoft Excel Solver.

It is concluded that sewer pipes (gravity and pressure pipes) in non-liquefaction zone may not necessarily suffer less damage than those in liquefactions zones. For SGP, the proposed fragility functions for LRI-2 and LRI-3 zones are more reliable and stable than those for LRI-0 and LRI-1 zones due to the limited number of CCTV inspections executed in Christchurch CBD. The developed fragility functions for SGP in LRI-2, LRI-3 and NLO zones can be directly applied in quantitatively estimating earthquake-induced physical damage to sewer pipes given a ground motion level for the preparation of rebuilding program. Furthermore, they can assist in seismic risk mitigation of sewerage pipelines before earthquakes.

A comparison study was conducted on the proposed fragility functions of SPP and existing WPP, showing a reasonably good agreement on the RRs of AC sewer pressure pipes. However, it is shown that the fragility functions defined for potable-water pipelines slightly underestimate the number of repair undertakings on those pipes due to the large number of SPP nearby waterways damaged by severe liquefaction and associated lateral spreading during the Canterbury earthquakes. By comparing DR of AC gravity pipes and RR of AC pressure pipes, it is concluded that fragility functions derived for SPP underestimate physical damage to SGP.

ACKNOWLEDGEMENT

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Identifying Seismic Vulnerability Factors for Wastewater Pipelines after the Canterbury (NZ) Earthquake Sequence 2010–2011

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Abstract
The 2010 and 2011 Canterbury earthquake sequence left Christchurch City's wastewater network severely damaged. This study undertakes an investigation into the performance of buried wastewater pipes after the Canterbury earthquakes adopting and integrating two different approaches, namely: 1) a statistical analysis of the repairs undertook by the pipes; 2) the classification of the observed vulnerabilities after field inspections. The aim is to assess how physical factors, including geometrical and constructive characteristics of pipes, along with the sustained seismic permanent ground deformation, have all contributed to the pipes' seismic performance. A database combining pipe physical factors, along with ground deformation measures, and repair rates (defined as number of repairs per km) was created and analyzed through statistical analysis, regressing repair rates against ground deformation respect to the physical factors. Then, the main factors identified via field visual inspections, are summarized. Finally the outputs from the two different approaches were comparatively discussed in the paper. The possibility to use the findings from this paper for the preliminary seismic vulnerability evaluation (e.g. screening and relative ranking) of buried wastewater pipelines is finally discussed in the paper.

INTRODUCTION

Sewage, widely recognized as wastewater, includes all the used water collected in internal drains from homes, businesses and commercial and industrial properties, such as water from sinks, basins, tubs, toilets, washing machines and dishwashers. A sewerage system is a complex reticulation network through which wastewater is conveyed from the above-mentioned internal drains to treatment plants by an underground pipeline network and associated assets. Because most of sewer components are buried under the earth and since wastewater pipes are usually designed only to resist soil loading they are vulnerable to transient ground motion and earthquake-induced permanent ground deformations and liquefaction. The highly probable earthquake-induced damage to wastewater pipes and other buried components can in turn reduce or cause a total loss of functionality for the sewerage system. The partial or total failure of a sewerage system in a post-disaster situation can have a dramatic impact on the community's resilience, creating the potentiality for severe public and environmental health hazard.

Examples of partial or total loss of functionality for sewerage systems following earthquakes can be identified worldwide. Approximately 150 km of sewer pipes and 2700 manholes were damaged to varying levels in the Hanshin event, Japan (1995). The moment magnitude (Mw) 7.4 Turkey earthquake (Izmit, Oct. 19. 1999) had serious impact on Izmit wastewater system, which used to have capacity of 10,500 liters per second but reduced to 30,000 liters per day due to earthquake effects. Tohoku earthquake (Mw= 9.0) in 2011 damaged 63 treatment plants, of which 48 totally lost their functionalities and seriously affected 101 sewer pump stations, of which 79 were out of service immediately after the event. The Canterbury earthquake sequence (CES) 2010-2011 caused significant damage to the Christchurch wastewater network and on 2nd April 2011, was deemed to be 'on the brink of failure' (Eidinger and Tang 2012), when Christchurch city wastewater treatment was operating at 30% of capacity, and the system was leaking 40 million liters per day into backyards and water courses, due to earthquake-induced damage to pipes (Figure 1). Temporary sewage facilities such as chemical and portable toilets were used for several months in Christchurch after the 22nd February 2011 earthquake to relieve strain on the wastewater system. (Kongar et al. 2015).

Despite the fragility of the wastewater network and the significance of the direct and indirect consequences that the loss functionality of such a service might have on large communities, methods and approaches for rapidly screening and assessing the seismic vulnerability of wastewater system (and other infrastructure systems generally speaking), are under-represented in the international literature with respect to buildings, for which several country-specific rapid screening approaches for seismic hazard are available (e.g. FEMA 154, 2002). This paper aims to advance practices towards the definition of a seismic vulnerability screening procedure for wastewater pipes in particular. Towards that, the proposed study undertakes an investigation into the performance of buried wastewater pipes after the Canterbury earthquakes, by combining statistical analyses and field observations. The aim is to assess how different factors have contributed to the seismic vulnerability of buried pipes, including: i) physical factors, such as geometrical and constructive characteristics of pipes (i.e. pipe material, diameter, length and age, depth of burial, joints type); ii) the sustained seismic permanent ground deformation; iii) other issues, such as installation quality, aging issues, and maintenance issues.

a) b)

Figure 1: Example of pipe failures after the Canterbury earthquakes: a) brick barrels concrete "egg" crack and silt ingress; b) reinforced concrete pipe joint failure with rubber ring exposed *(photo credit: SCIRT, used with permission).*

The following is structured such that, some background information on the Canterbury earthquakes and on the investigations undertaken by different authors on the performance of Christchurch buried pipes are provided firstly. Secondly, the findings in terms of factors affecting the seismic vulnerability of wastewater pipes after the Canterbury earthquakes, from the statistical analysis are presented, along with the data and approach used to obtain them. Finally, the main factors influencing seismic vulnerability of wastewater pipes identified after visual inspections are summarised and related to the outputs resulting from the statistical analysis. Considerations on the possibility that asset managers could use the proposed list of vulnerability factors, for screening and ranking the relative seismic vulnerability of existing wastewater pipes is discussed in the conclusions. The final aim being the identification of the pipes, whose substitution should be prioritized to mitigate the seismic risk for the network, to be considered and addressed as part of business as usual asset management plans.

BACKGROUND INFORMATION

The CES in New Zealand is notable for five major events, namely September 4, 2010 earthquake (M_W=7.1), December 26, 2010 swarm (max. M_W=4.7), February 22, 2011 (M_W=6.2) and June 13, 2011 (M_W=6.0), December 23, 2011 (M_W=5.9). The first main event on the 4th September 2010 and aftershock sequence struck close to the town of Darfield on South Island of New Zealand, 30 km west of Christchurch. Christchurch is the second largest urban centre in New Zealand and the Darfield earthquake was the first large earthquake to strike close to an urban centre in New Zealand since the Hawke's Bay earthquake of 1931(Giovinazzi et al., 2011). There were no fatalities and only two serious injuries (Wood et al., 2010). Peak horizontal accelerations (PHAs) were in the range 0.3 g to 0.82 g. Peak ground velocities (PGVs) exceeded 1 m/s. The second main event on 22nd February 2011 (M_W 6.2) struck the city of Christchurch and it was very shallow at about 5-6 km and the epicentre causing extremely high ground accelerations across the city. The earthquake caused 185 causalities, 8,600 injuries and widespread damage to buildings and lifelines. This event occurred while the Canterbury region was still recovering from the Darfield earthquake on 4th September, with many structures suffering from compounded damage. The highest PHA recorded was 1.41g (Bradley & Cubrinovski, 2011). All the main events of the earthquake sequence events, but in particular the 22nd February earthquake, caused unprecedented levels of liquefaction, throughout the southern and eastern suburbs of Christchurch (Yamada et al. 2011) alongside the Avon River1. The liquefaction resulted in settlement, lateral spreading, sand boils, and a large quantity of ejected silt mud and water ponding on the ground surface.

Several studies have been carried out, since February 2011, into the performance of buried pipe in Christchurch. O'Rourke et al. (2012) analyzed the water distribution piping system and its 1645 main repairs. They noted 10 to 30 times more pipe failures in zones with observed liquefaction, and also found repair rates for asbestos cement and cast iron pipes were 4 to 5 times greater than for polyvinylchloride (PVC) pipes, which in turn were 2 to 4 times greater than for modified PVC pipe. Cubrinowsky et al. (2014) presented a preliminary analysis of the performance of Christchurch City's potable water, wastewater and road networks through the 2010-2011CES. In particular, the report is mainly focused on the analysis of liquefaction effects on the damage and performance of the buried pipes of different materials. By processing damage data, they found that the earthenware and concrete pipes for the unpressurised waste water pipes and galvanized iron for the water pipes, resulted more vulnerable to the earthquake, being the ones where the highest repair rates were found amongst. In particular for the analysis of wastewater pipes Cubrinowsky et al. (2014) used data on repairs per pipe for the period 5 September 2010 to 5 June 2013. The study proposed in this paper aims to identify and analyse the influence of different factors further than the pipes material (as specified in the introduction) on the seismic performance of wastewater pipes.

PHYSICAL IMPACT OF CES ON WASTEWATER PIPES

Christchurch wastewater system includes over 1700 km of gravity fed pipes. The range of material includes: reinforced concrete with rubber rings (RCRR), earthenware (EW), polyvinylchloride (PVC), unplasticised (UPVC) and modified (MPVC) polyvinylchloride, unreinforced concrete (CONC), cast iron (CI), asbestos cement (AC), polyethylene (PE), high density (HDPE) and medium density (MDPE) polyethylene (Kongar et al., 2015). Data on Christchurch wastewater network and on the repairs carried after the Canterbury earthquake on which this study is based, were supplied by The Stronger Christchurch Infrastructure Rebuild Team (SCIRT) in the form of two Graphical Information System (GIS) shaplefiles, namely: 1) Christchurch City Council wastewater (CCC-WS) pipe shapefile; and 2) Citycare wastewater repair (Citycare-WS-R) shapefile. CCC-WS pipe database included the following information for each pipe: a unique identifier (ID); material; length; diameter; age; and depth; CitycareWS-R database contained information on recorded repairs (for the period from 22 February 2011 to 3 July 2012) ascribed to each pipe record.

Permanent ground deformation values were estimated from the Liquefaction Resistance Index (LRI) map provided by Cubrinovski et al. (2011) that categorizes Christchurch City into 5 LRI zones (from 0 to 4) according to the range of lateral displacement and ground settlement observed in each (Table 1). The average lateral displacement and ground settlement for each zone were combined using vector addition to create a permanent ground deformation (PGD) value for each LRI zone.

Table 1. PGD values used in this study for each LRI zone defined by Cubrivnoski et al. (2011, 2014)

LRI Zone	Ground settlement (mm)	Lateral Displacement (mm)	PGD (mm)
0	>500	>400	640
1	250-500	200-400	480
2	50-250	40-200	192
3	20-50	20-40	46
4	<20	<20	14

The hazard map, the pipe network and pipe repairs shapefiles were overlaid. PGD values and repairs were assigned to each individual pipe within the network. Repair rates (defined as number of repairs per km) were then evaluated respect to each pipe material and diameter ranges and for different levels PGD. For the evaluation of repair rates, the study area was restricted to the boundaries of the Liquefaction Resistance Index (see Figure 2) as the PGD values estimated were limited to this source. Additionally, only pipes under Citycare maintenance contract were studied; private lines were excluded. Pipes with undefined ages were removed from the study. This excluded around 10% of the 35,500 pipes in the network. As this was a small portion of data, the effect of this action was deemed negligible.

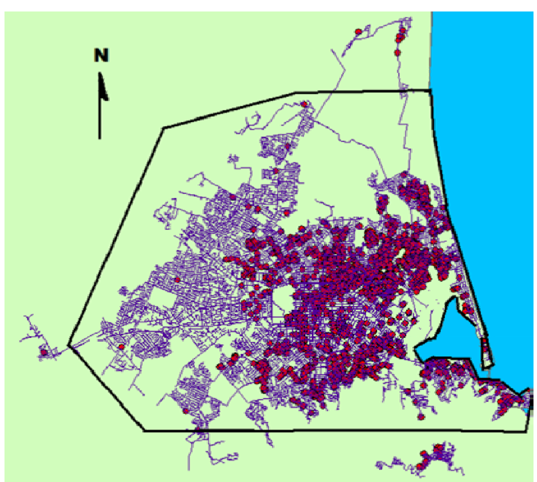

Figure 2. Christchurch wastewater network and repairs on boundary of studied area.

Regarding the influence of pipe material, from a preliminary analysis of the data (Table 2) it was observed that the older and often more brittle pipes such as concrete, earthenware, cast iron and reinforced concrete suffered more damage than PVC and PE on average across the network as also observed by Cubrivnoski et al. (2014).

Table 2 Observed repair rate for wastewater pipe materials following the February 2011 earthquake.

Pipe Material	Length (km)	% of total	Number of repairs	Repair Rate (RR)
EW	332	22.3	982	2.96
CONC	93	6.3	205	2.20
CI	40	2.7	55	1.37
RCRR	612	41.1	651	1.06
PE (all)	17	1.1	16	0.94
AC	115	7.7	100	0.87
PVC (all)	279	18.8	69	0.25
Total	1488		2078	1.40

The influence of pipe diameter on pipe performance was also investigated (Figure 3). The amount of damage sustained decreased as the pipe diameter increased for all pipe materials. However, this outcome is based on a small sample set of large diameter pipes as less than 3% of the pipes in the data set had a diameter greater than 750mm.

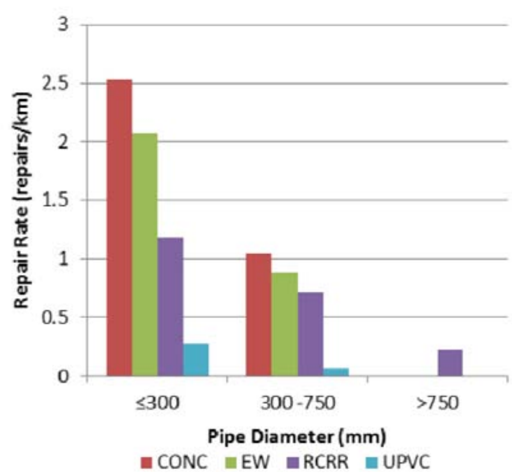

Figure 3. Influence of pipe diameter on damage sustained.

Further, the influence of different levels of sustained permanent ground deformation on repair rate across a wide range of pipe materials was analyzed (Figure 4). For each PGD class identified in the map, the number of repairs was divided by pipe length located in the given PGD class respect to each material. The effect is strongly non-linear with the repair rate rising greatly with increased PGD.

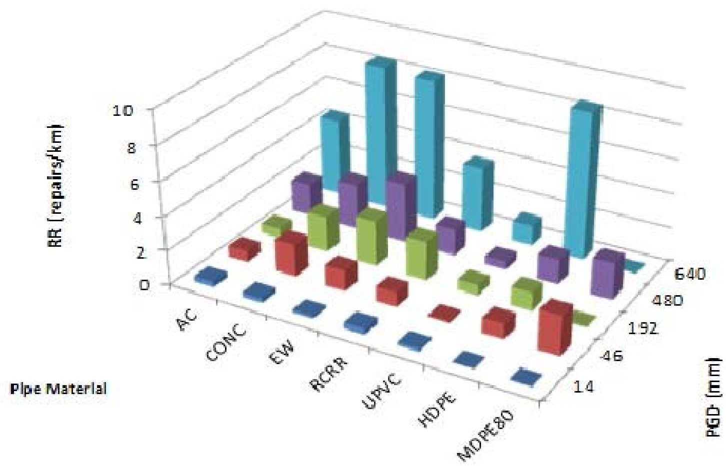

Figure 4. PGD versus repair rates respect to various pipe materials.

A multiple linear regression analysis was then performed in order to evaluate the correlation between pipe parameters and seismic damages of pipelines. Each pipe was classed as a failure or a non-failure, and this binary variable was then regressed against various parameters. Parameters analyzed were: age, depth, length, diameter, material and PGD. Each pipe material was assessed individually to determine the key factors contributing to its damage exception made for MDPE, PE, PVC and MPVC as there were too few instances of failure to deduce a meaningful relationship.

While it was of interest to determine the most prominent parameter contributing to a pipe failure, establishing which parameters had little or no effect on the performance of a pipe material was of equal interest.

The T-Test was then performed to check the significance of individual regression coefficients assuming a 0.95 significance level, i.e., the relationship between a given parameter and failure was considered to be of no significance if the P-value for the null hypothesis of no linear effect was greater than 5%. Table 3 provides the t-statistics for those relationships with P-values less than 5% (NS indicates 'No Significance').

Table 3. T-statistics for Linear Regression of Pipe failure versus various factors.

	T-Stat by Pipe Material					
Parameter	**AC**	**CONC**	**EW**	**HDPE**	**RCRR**	**UPVC**
Diameter	-4.9	-4.0	NS	NS	-9.0	NS
Age	NS	NS	NS	NS	NS	NS
Depth	6.7	-2.5	NS	NS	5.5	5.4
Length	NS	-2.3	7.7	NS	5.9	NS
PGD	4.3	5.1	14.3	2.6	12.5	4.1

Most Significant	Depth	PGD	PGD	PGD	PGD	Depth

Analysis indicates that:

- depth and PGD are the most significant contributors to pipe failure;
- for the pipe materials assessed (i.e. AC, CONC, EW, HDPE, RCRR and UPVC), there is a significant and positive correlation between PGD and pipe failure and therefore a strong relationship between increasing PGD and repair rate;
- where significant, the correlation between diameter and failure is negative indicating that pipe failure decreases with increasing pipe diameter;
- there is no significant correlation between age and failure;
- the correlation between pipe length and failure is largely inconsistent;
- the correlation between depth and pipe failure is not significant for two cases (EW and HDPE). However, for three cases (AC, RCRR and UPVC) there was a very strong positive correlation between depth and failure.

OBSERVED SEISMIC VULNERABILITY OF PIPES FROM FIELD INVESTIGATIONS

Information on observed damage experienced by different pipe typology of the water and wastewater systems during the CES, were collected and collated by Black (2012). Black (2012) observed that the main contributions to the poor performance of pipes, or in other words the main factors affecting the seismic vulnerability of buried pipelines, when subjected to liquefaction-induced PGD, included:

- pipe characteristics - e.g. pipe and joint material type, pipe diameter, age of construction;
- manufacturing quality - e.g. poor quality of installation, that represented a bearing on the seismic performance for all pipe materials;
- pre-existing issues - e.g. corrosion, aging and maintenance issues.

As for the first point, i.e. pipe characteristics, Table 4 summarizes the observed seismic vulnerability, for different pipe materials, joint types and diameters (when such information is available) according to three qualitative vulnerability classes namely, “high”, “medium” and “low”.

Table 4. Observed seismic vulnerability for wastewater pipes by pipe material, joint type and pipe diameter (adapted from Black 2012).

Pipe Material	Joint Type	Diameter	Observed Vulnerability
Brick and stone barrels	Lime mortar jointing		High
Ceramic pipes	Mortar		High
Ceramic pipes	Rubber ring		High
Unreinforced concrete pipes	Rubber ring		High

Reinforced concrete pipes (old)	Rigid lead joints		High
Reinforced concrete pipes (old)	Rubber ring	Small	High
Reinforced concrete pipes	Rubber ring	Large	Medium
Cast iron (CI) pipes	Rigid, run-lead joints		High
Cast iron (CI) pipes	Rubber ring		High
Asbestos cement (AC) pipes		≤ DN 150	High
Asbestos cement (AC) pipes	Rubber ring	> DN 200	High
Steel	Screwed	≤ DN 50	High
Steel	Lead joints		High
Steel pipes (concrete lined steel CLS)	Rubber ring joints		Medium
Steel pipes (concrete lined steel CLS)	Full strength welded joints		Low
Glass reinforced plastic (GRP)	Butt and strap joints		Medium
Glass reinforced plastic (GRP)	Rubber ring		Medium
Ductile Iron (DI)	Rubber ring		Medium
Ductile Iron (DI)	Locking rings (TytonLok)		Low
Ductile Iron (DI)	Seismic joints		Low
PVC-U – Polyvinylchloride	Solvent Cement Joints		Medium
PVC spigot and socket pipes	Rubber ring joints		Medium
Polyethylene (PE) pipes with structured walls	Rubber ring joints		Medium
PE pipes (first generation-type 5 HDPE resins)	End-load bearing joints		Medium
PE 80B or PE 100 pipes	End-load bearing mechanical joints		Low
PE 80B or PE 100 pipes	Electro-fusion joints		Low

In general terms, with reference to pipe material and joint type, Black (2012) observed that brittle pipe materials with rigid joints proved their high vulnerability, due to their "weak links", where the greatest number of earthquake-induced failures occurred. As for the diameter, the larger the pipes, the less susceptible they seemed to earthquake damage. Larger diameter pipes (even of brittle materials) have significantly greater beam strength, or ability to resist deflection, than smaller pipes making them less susceptible to bending and circumferential cracking failures Black (2012).

As for the manufacturing quality, according to Black (2012) factors increasing the seismic vulnerabilities of pipes, included:

- pipe manufacturing faults (pipe quality);
- poor workmanship during handling and installation; and

- mis - application (design issues).

Any of them or any combination of them seemed to have contributed to pipe failures (Black 2012) following the Christchurch earthquakes.

As for pre-existing issues, Black (2012) observed that:
- "conventional" pipe materials, such as e.g. AC, CI and DI and steel all might suffer from a range of deterioration mechanisms that reduce their strength and make them progressively more vulnerable to failure as they deteriorate. Modern corrosion protection methods might delay the onset of corrosion but they must remain effective for the design life of the pipe, usually at least 100 years.
- plastics pipes, such as PVC and PE do not suffer from corrosion but other mechanisms might affect their vulnerability, including: i) chemical break-down of the polymer structure; ii) break-down of the stabilizers, in the older PE resins (type 5 HDPE in New Zealand).

CONCLUSIONS

This study has identified the main trends in wastewater pipe failure in Christchurch and the significant contributors to pipe vulnerability under seismic loading. Statistical analysis has shown that the most significant factors in pipe failure under earthquake loading are the sustained PGD, the pipe buried depth and the pipe material. Both PGD and depth of these parameters are strongly related to liquefaction susceptibility. As for the pipe material, brittle pipes present in the Christchurch wastewater network, such as AC, CI, EW and RCRR suffered higher amounts of damage than the plastic pipe materials, i.e. PVC and PE. This study has observed that PVC pipes suffered less damage than PE. This is contrary to industry belief and should be investigated further (the data available on these materials within this study were limited and that might have biased the results). As for the pipe diameter, where significant, the correlation between diameter and failure was found to be negative, indicating that pipe failure decreases with increasing pipe diameter. Unfortunately data on joint type were not available; therefore it was not possible to check their role in the seismic performance of pipes with the statistical analysis approach.

All the results from the statistical analysis resulted in line with the ones from field inspections, collected and summarized in the paper. From observations on the field it was confirmed that brittle pipe materials with rigid joints proved to be the most vulnerable. Similarly, as far as the pipe diameter was concerned, the larger the pipes, the less susceptible they seemed to be to the earthquake damage. A vulnerability tables was produced out of field observations, classifying coupled pipe material, joint types and diameters into vulnerability classes. The proposed classification is all in line with the outcomes from the statistical analysis.

The approach taken to cross-calibrate statistical analysis with observations and expert-based evidences and vice-versa seems to be very effective towards the definition of scorecard approaches and/or vulnerability indexes and/or rapid screening approaches (as the ones widely available for buildings e.g. FEMA 154) specific for buried pipelines. Figure 5 presents an example of a possible outputs from the analysis presented in this paper, namely a vulnerability matrix that identify the vulnerability of pipes according to three qualitative classes (green=low vulnerability; yellow= medium vulnerability; red=high vulnerability). The definition of vulnerability index and of a rapid screening approach based on the findings of this paper will be present as part of a future publication by the same authors.

	Consequences		
Hazard	**Low (RR<0.5)**	**Medium (0.5<RR<2)**	**High (RR>2)**
Low (PGD[mm]<=14)	UPVC HDPE MDPE AC PVC	CONC EW RCRR CI	
Medium (46<PGD[mm]<480)	UPVC PVC	AC HDPE MDPE CI	EW CONC RCRR
High (PGD[mm]>480)	PVC	UPVC HDPE	AC CONC EW RCRR CI

Figure 5. Example of vulnerability matrix for pipes (green=low vulnerability; yellow= medium vulnerability; red=high vulnerability).

ACKNOWLEDGEMENTS

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Shaking Table Test for Axial Behavior of Buried Inner Rehabilitated Pipes Affected by Aging Pipes in Liquefied Ground

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Abstract

In Japan, the main irrigation pipeline network is increasingly getting older and losing its function due to deterioration. The concept of in-situ rehabilitation for aging pipelines, which can improve the performance of a damaged pipe by installing new pipeline inside of the existing aging pipeline, has gained increasing attention. However, the seismic mechanical behavior of pipeline rehabilitated by this method is not substantially clear. In this study, to clarify the effect of damaged outer pipe on the inner pipe, shaking table tests were conducted for the buried pipe in liquefied ground. To model the inner pipe, polyvinyl-chloride (PVC) and polyethylene (PE) pipe were used. To model the outer aging pipe, different types of concrete pipes in length were used. The test results indicated that the amplitude of the bending strain in PVC pipe sharply increased due to the stress concentration at the gap between outer pipes. Additionally, the deflection mode of the pipe was categorized into two main types; bow-shaped deformation and pendulum-shaped deformation.

INTRODUCTION

In Japan, pipelines have been extended as irrigation canals. Open channels are replaced with pipelines to save of irrigation water, to improve of water quality, and to prevent of water accidents. The length of the main canal is approximately 49,900 km, and the total length of the network extends more than 400,000 km

(M.A.F.F., 2012). Approximately 30% (13,578 km) of the irrigation canal is pipelines.

In recent years, the main irrigation pipeline network has lost some functionality because of deterioration. Pre-stressed concrete (PC) pipe and reinforced concrete (RC) pipe were used as irrigation pipelines during the 1960s and 70s. The economic lives of the pipe are approximately 40 years. These pipes have already exceeded their expected economic lives (M.A.F.F., 2010) and have started deteriorating, causing water leakages or bursts, especially during earthquake.

The concept of in-situ rehabilitation, so-called "trenchless pipe rehabilitation," has gained increasing attention as a method to repair aging pipelines. With this method, inner pipes (rehabilitated pipes) are directly constructed or inserted into outer pipes (aging pipes) to improve the performance of aging pipes. The rehabilitated pipeline system comprises two pipes; an outer aging pipe and an inner rehabilitated pipe. **Figure 1** shows the diagram of a pipe after the rehabilitation method has been performed.

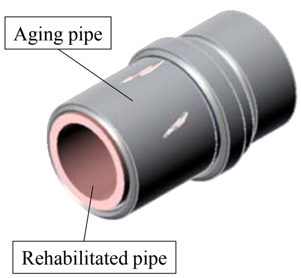


Figure 1. Pipeline after rehabilitation method has been performed.

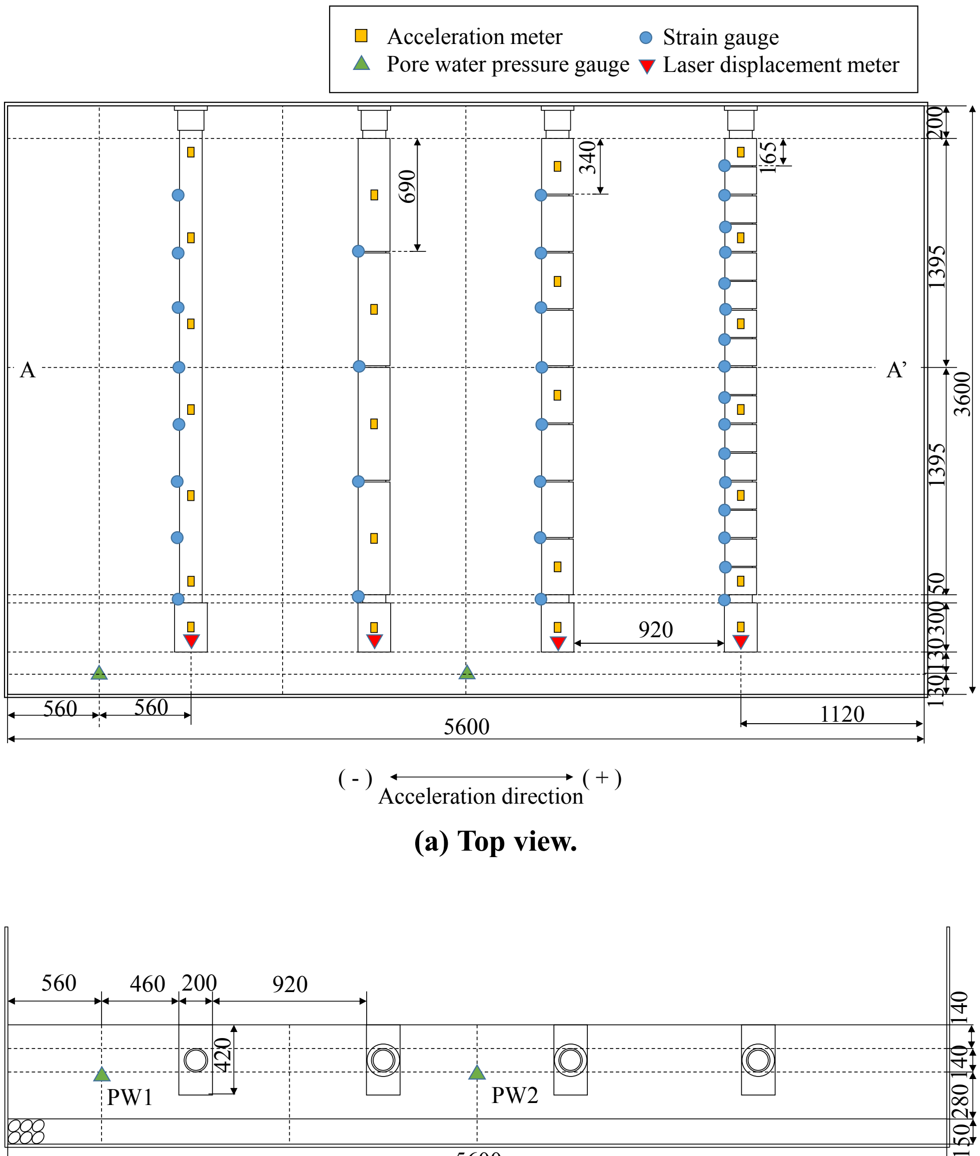
Many experiments have tried to verify the static behavior of the cross section of pipe that has been rehabilitated with this method. Gumbel et al. (2003) and Spasojevic et al. (2007) conducted centrifuge model tests using a four-hinged model pipe as the damaged outer pipe. Sawada et al. (2014) conducted the model experiments under different damage levels of the aging pipe and loading positions. Ono et al. (2014) conducted centrifuge model tests, showing the quantitative experimental results with different damage levels of the outer pipe.

The influence of damaged outer pipes on inner pipes in the axial direction during an earthquake is unclear. In our study, shaking table tests were conducted at the National Institute for Rural Engineering in Japan to verify the dynamic behavior of the axial direction of the inner pipe used for the method. To model inner pipes, polyvinyl-chloride (PVC) and polyethylene (PE) pipes, which were 3,040 mm in length and 140 mm in diameter, were used. To model outer pipes, different types of concrete pipes in varying lengths were used.

OUTLINE OF EXPERIMENT

Test equipment. The shaking table used for the test had plane dimensions of 6 m × 4 m, with the maximum loading capacity of 50 tf. Its excitation system was an electro-hydraulic servo. The test pit (5.6 m length, 3.6 m width, and 1.3 m height) as shown in **Figure 2**, was installed on the shaking table. At the bottom of the test pit, a bed of crushed stone was included to enhance the capabilities of pouring water into the ground.

For the model pipe, the bending strain in the axial direction, the horizontal acceleration and the horizontal displacement of the pipe end, and the pore water pressure in the model ground were measured. **Figure 2** shows the layout of measurements.



(a) Top view.

(b) Cross-section view.

Figure 2. Schematic layouts of model pipes and sensors.

Model ground. Kasumigaura-sand was used as a soil material for the model ground. **Table 1** shows the properties of Kasumigaura-sand. The unsaturated ground was adequately compacted to reach the target relative density of 50%.

Table 1. Properties of Kasumigaura-sand.

Density of soil particle ρ_s	2.72 g/cm^3
Maximum dry density ρ_{dmax}	1.58 g/cm^3
Minimum dry density ρ_{dmin}	1.36 g/cm^3
Maximum void ratio e_{max}	0.99
Minimum void ratio e_{min}	0.72
Relative density D_r	50%

Model inner pipe. PVC and PE pipes were used as inner pipes. Their properties are shown in **Table 2**. One end of the inner pipe was rigidly-connected to the test-pit wall, as shown in **Figure 2**. Lead shot and silica sand were inserted into the inner pipes to prevent the model pipe from floating during liquefaction. Both ends of the inner pipe were waterproofed. A concrete block weighting 50 kg was installed at the other end. When inertia forces were applied to the concrete block during shaking, it was expected that deflections of the pipe efficiently amplified. The block was hoisted by chains to avoid sinking in the liquefied ground.

Table 2. Properties of pipe.

	Length L (mm)	Thickness t (mm)	Outer Diameter D (mm)	Bending stiffness EI (kN · m)
PVC	3,000	4.1	140	15,354
PE	3,000	7.0	140	7,250

Model outer pipe. Concrete pipes were made to simulate RC pipes as outer pipes. These pipes had an inner diameter of 140 mm and a thickness of 27 mm. They were made in three-different lengths: with four segments (three joints), eight segments (seven joints), and 16 segments (15 joints) as shown in **Table 3**. They were divided in the axial direction as show in **Figure 3**. After curing for seven days, the unconfined compression strength of the concrete was 22.72 N/mm^2.

Table 3. Properties of concrete pipe.

Type	Length per one segment L (mm)	Density ρ (g/cm^3)
Three joints	690	2.28
Seven joints	340	2.13
15 joints	165	2.10

Figure 3. Concrete pipe.

Combination of inner pipe with outer pipe. The outer diameter of the inner pipe corresponded to the inner diameter of the outer pipe. Outer pipes were tightly fit to inner pipes. To prevent outer pipes from sliding, the pipe was restricted by a band. The gap between outer pipes was 10 mm and was covered by membranes to simulate a joint structure in the axial direction.

Procedure of experiments. Horizontal shakings were applied after backfilling. The model pipes were buried at a depth of 140 mm (1*D*). The ground was saturated from the bottom of the pit. Input acceleration generated a sinusoidal wave at a frequency of 2 Hz with the maximum acceleration of 500 Gal as shown in **Figure 4**. **Figure 5** shows the response of the horizontal displacement of the shaking table.

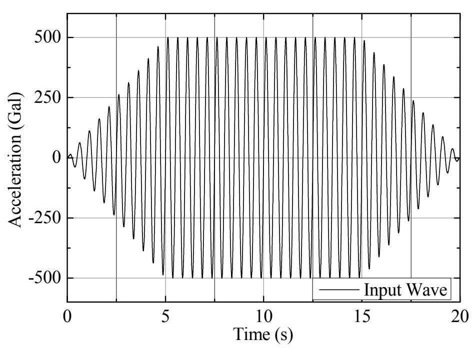


Figure 4. Response of acceleration of shaking table.

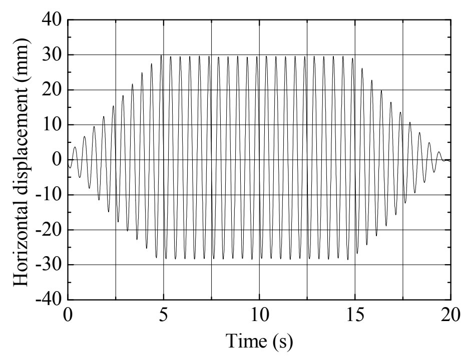


Figure 5. Response of horizontal displacement of shaking table.

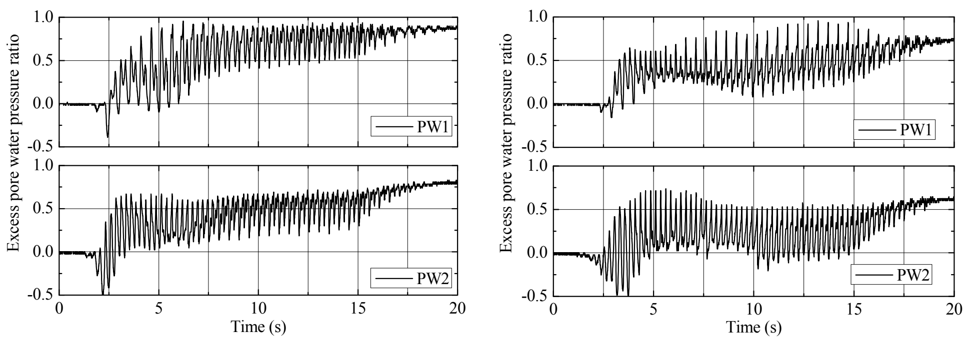
Experimental cases. Experiments were conducted using different types of inner pipe and outer pipe as shown in **Table 4**.

Table 4. Experimental cases.

Series	Case	Type of inner pipe	Type of outer pipe
A	PVC00	PVC	N/A
	PVC03		Three joints
	PVC07		Seven joints
	PVC15		15 joints
B	PE00	PE	N/A
	PE07		Seven joints
	PE15		15 joints

RESULTS AND DISCUSSIONS

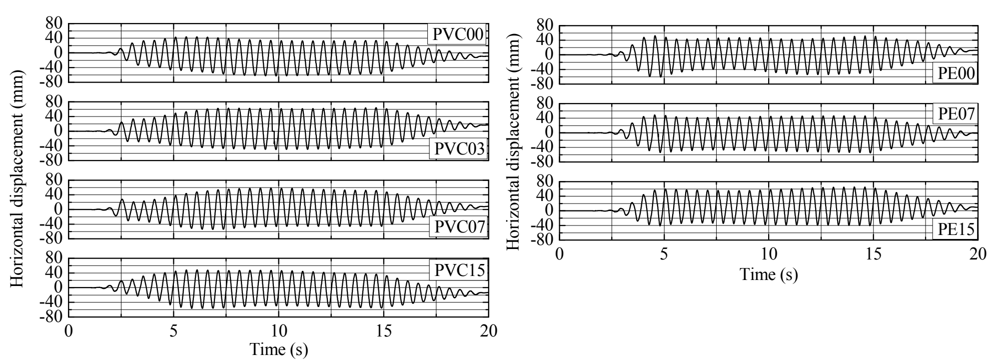
Liquefaction of ground. Liquefaction occurred during the shaking, judging from the excess pore water pressure ratio as shown in **Figure 6** in both series. An excess pore water pressure ratio was calculated as the ratio of the initial effective stress to the change of the excess pore water pressure. The excess pore water pressure ratio increased at approximately 2.5 seconds from the start of the shaking. However, the excess pore water pressure ratios did not reach 1.0. It indicates that the ground did not liquefy completely.



(a) Series A. (b) Series B.

Figure 6. Response of excess pore water pressure ratio.

Response of horizontal displacement of end of pipe. The influence of the number of joint structures on the horizontal displacement of the pipe was extremely small. **Figure 7 (a)** shows the response of the horizontal displacement of the pipe end in Series A. In every case, the end of the pipe moved horizontally due to the liquefaction as shown in **Figure 6**. The horizontal displacement of the pipe end increased with the increment of acceleration of the shaking table. The difference between the maximum displacement and the minimum for PVC00, PVC03, PVC07, and PVC15 is 107 mm, 114 mm, 112 mm and 105 mm, respectively. As shown in **Figure 7 (b)**, the horizontal displacement in Series B was also similar to that in Series A.

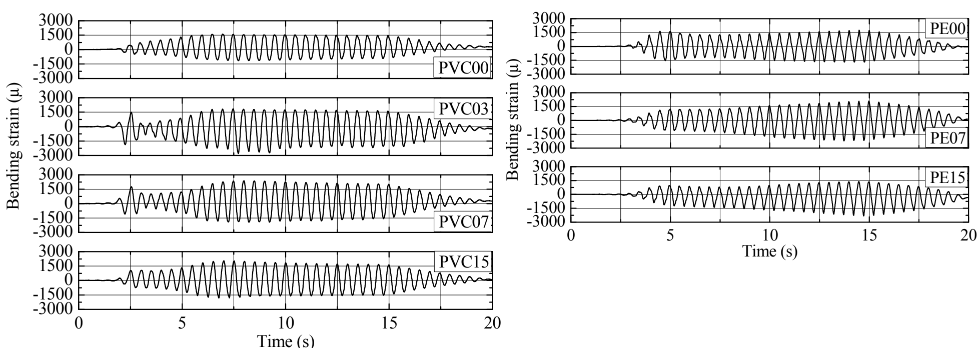


(a) Series A. (b) Series B.

Figure 7. Response of horizontal displacement of end of pipe.

Response of bending strain of inner pipe. The amplitude of the bending strain that PVC pipe had on the outer pipe was large when the liquefaction occurred. **Figure 8 (a)** shows the response of the bending strain at the center (A-A' section as shown in **Figure 2**) in Series A. In PVC00, the amplitude of the bending strain increased, depending on the increasing amplitude of acceleration of the shaking table. For PVC00, PVC03, PVC07 and PVC15, the amplitude reached approximately 500 μ, 1,500 μ, 1,500 μ and 1,000 μ, respectively, at approximately the time when the ground liquefied. Tease results indicate that the outer pipe restricted the deformation of inner pipe when it was moved. Therefore, the stress was concentrated at the gaps between the outer pipes.

On the other hand, the concentration at the gaps between the outer pipes did not occur in the PE pipe. **Figure 8 (b)** shows the response of the bending strain at the center in Series B. The wave form for PE00 showed no difference from the wave form for PE07 and PE15 at the time when the ground liquefied. In addition, the amplitude of the bending strain increased from 7.5 seconds to 15 s in every case, although the amplitude of the acceleration of the shaking table was constant at 500 Gal. The creep of the PE pipe is considered to cause this result.

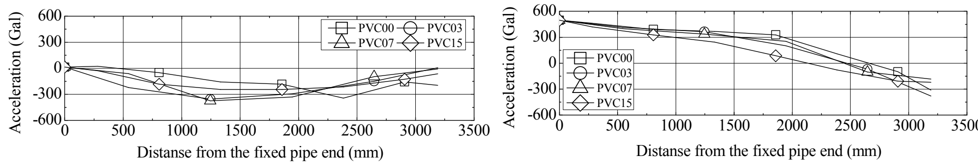


(a) Series A. **(b) Series B.**

Figure 8 Response of bending strain of inner pipe.

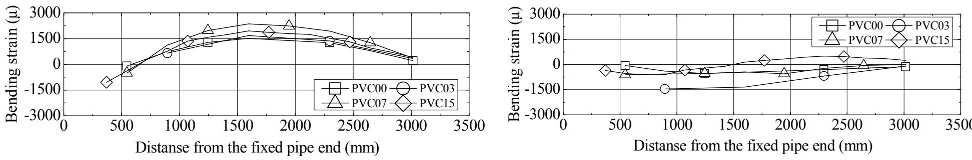
Deflection mode of inner pipe. The deflection mode of the model pipes was categorized, in every case, into two main types: bow-shaped and pendulum-shaped deformation. **Figure 9** shows the distribution of the acceleration, and **Figure 10** shows the distribution of the bending strain in Series A. The displacement of the shaking table is 0 mm (t = 8.0) in **Figure 9 (a)** and **Figure 10 (a)**, and the displacement is the maximum at approximately 30 mm (t = 8.125) in **Figure 9 (b)** and **Figure 10 (b)**. The distribution of the acceleration looks bow-shaped as shown in **Figure 9 (a)**, and the bending strains at the rigid ends and around the centers developed as shown in **Figure 10 (a)**. However, the model pipes were deformed like a pendulum-shaped, judging from **Figure9 (b)** and **Figure 10 (b)**. The bending strain of a bow-shaped deformation is clearly larger than that the bending strain of a pendulum-shaped deformation.

Deflection mode of PE pipe is similar to that of PVC. **Figure 11** shows the distribution of the acceleration, and **Figure 12** shows the distribution of the bending strain in Series B. When the displacement of the shaking table was 0 mm, the PE pipe was deformed similarly to the PVC pipe as shown in **Figure 11 (a)** and **Figure**



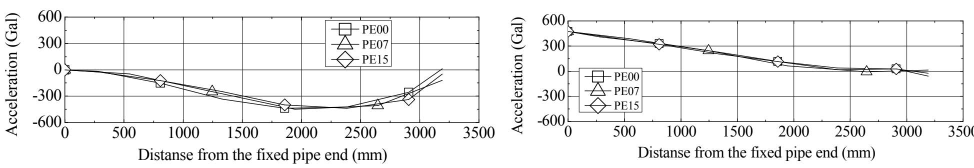
(a) t = 8.0. **(b) t = 8.125.**

Figure 9. Distribution of acceleration of model pipe in Series A.



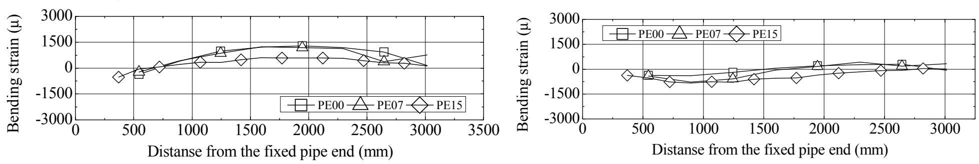
(a) t = 8.0. **(b) t = 8.125.**

Figure 10. Distribution of bending strain of model pipe in Series A.



(a) t = 8.0. **(b) t = 8.125.**

Figure 11. Distribution of acceleration of model pipe in Series B.



(a) t = 8.0. **(b) t = 8.125.**

Figure 12. Distribution of bending strain of model pipe in Series B.

12 (a). In addition, the PE pipe was also deformed like pendulum-shaped when the displacement of the shaking table was approximately 30 mm as shown in **Figure 11 (b)** and **Figure 12 (b)**.

CONCLUSIONS

The shaking table tests were conducted to verify the dynamic behavior in the axial direction for the trenchless pipe rehabilitation method during liquefaction. The following points were clarified from the test results:

1. The influence of the number of joint structures on the horizontal displacement of the pipe was extremely small.
2. The amplitude of the bending strain of PVC pipe having the outer pipe was large when liquefaction occurred. It was considered that the stress concentrated at the gaps between the outer pipes.
3. The deflection mode of the pipe was categorized into two main types: bow-shaped and pendulum-shaped deformation. Compared with the bending

strain of a pendulum-shaped deformation, the bending strain of a bow-shaped deformation was large.

ACKNOWLEDGEMENT

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Design and Fabrication Requirements of a High-Pressure Steel Pipeline

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Abstract

The desalination project with 54 million gallon per day (MGD) capacity in Carlsbad, California, which is under construction, includes a 54-inch diameter, 10-mile long high-pressure steel pipeline for the conveyance of the desalinated ocean water from the treatment plant to the San Diego County Water Authority's water system. The steel pipeline is lined with cement-mortar and coated with polyethylene tape and cement-mortar topcoat. The design criteria and parameters for the pumped flow system pipeline, which included seismic analysis, were stringent due to the operating conditions. The steel pipeline and its field double lap-welded joints were designed for a maximum design and surge pressures of 537 psi and 662 psi, respectively. The preliminary lap-welded joint design was based on ASME's joint efficiency factors and the final joint design was based on the joint stress analysis. The preliminary and final design criteria, which included limiting the maximum design circumferential stress at surge pressure to 50% of the minimum yield strength of the steel, will be reviewed and presented. The steel cylinders, with thicknesses varying from 0.50 inch up to 0.70 inch for 64% of the total length of the pipeline, were fabricated from coiled steel by the spiral forming and welding process. The remainder of the steel cylinders, with thicknesses varying from 0.72 inch up to 0.90 inch, were fabricated from plate steel by the rolling and welding process. The weld-seam fabrication tolerances and allowable repairs were more stringent than AWWA C200 standard requirements. In addition, 100%of the weld seams of the steel cylinders were radiographically tested. The fabrication and testing requirements of the steel cylinders will be reviewed and presented.

INTRODUCTION

The Carlsbad desalination project which is the largest ocean water-to-drinking water plant in the Western Hemisphere includes a 54-inch diameter, 10 mile long high pressure steel conveyance pipeline (Carlsbad Conveyance Pipeline) to the San Diego County Water Authority's (SDCWA) water system. The $1 billion design-build project by Poseidon Resources is expected to deliver 54 MGD of drinking water for San Diego County when it is completed in 2016. This high-pressure conveyance pipeline, which is under construction, will be completed in early 2015. The SDCWA's Design Manual for Facility Design Guide was used as the main guide during the design of the 54-inch diameter pipeline. Additional project specific requirements were also added during the review processes with the City of Carlsbad initially and with the SDCWA during the final designs. The initial delivery point through the SDCWA's water system was changed to the Twin Oaks Water Treatment Plant Clear Wells which resulted in approximately 85 foot higher static head and consequently higher pumped heads during the final designs. The pump station at the desalination plant includes a surge tank for surge pressure mitigation; in addition the air vacuum valves installed in the existing water system will also serve as a secondary surge mitigation control.

Figure 1 shows the pipeline's construction activity near the pumping station of the desalination plant.

Figure 1. Construction Activity near the Startup of the Pipeline. A 54-inch diameter pipe section with 30-inch diameter outlet and single-plate crotch plate reinforcement is ready for installation.

The 54-inch diameter high pressure steel pipeline was fabricated in conformance with SDCWA's General Conditions and Standard Specifications as well as project related modifications. The steel pipe was lined with 0.50 inch cement-mortar and coated with polyethylene tape and 1 inch cement-mortar overcoat in conformance with SDCWA's Standard Specifications.

INITIAL CARLSBAD CONVEYANCE PIPELINE DESIGNS

Hydrostatic Pressure Design and External Load Checks

The preliminary designs of the steel pipe for the Carlsbad Conveyance Pipeline were conducted for the internal design pressures established by the project hydraulic profiles and were checked for the external earth and live loads and external pressures. The hydrostatic pressure designs were conducted in conformance with AWWA M11 using the Barlow formula and the minimum wall thickness of 0.50 inch required by the SDCWA's design guidelines. Since the hydraulic profiles for operating and surge conditions were based on the 30% Design Bid Set, a 50 psi additional pressure was added to the maximum operating pressures in the hydraulic profiles for determining the design pressures. Six classes of pipe starting from 250 up to 500 psi in 50 psi increments and one 525 psi class were established from the hydraulic profiles of the conveyance pipeline. A design stress of 18,000 psi was used for pipe classes 250 and 300, and 20,000 psi for the higher classes. This resulted in a maximum cylinder thickness of 0.725 inch for class 525. Seismic analysis was not performed during the preliminary designs.

The allowable external earth and live loads were determined by limiting the pipe deflections to 1% required by the SDCWA's design guideline and using the AWWA M11, Equation 6-5, modified Iowa Deflection Formula. A check for earth and vacuum pressure against allowable buckling pressure per AWWA M11, Equation. 6-7 was also performed.

In addition, sensitivity analysis of the native soil stiffness to pipe embedment stiffness, which was requested by of the City of Carlsbad, was performed to determine if the in-situ soils impact the pipe embedment stiffness. The in-situ soil stiffnesses were evaluated from the project geotechnical soil borings for the SPT (Standard Penetration Test per ASTM D1586) values and corresponding estimated E′ values per AWWA Manuals M45 and M55.

Summary of Design Conditions and Criteria

<u>Minimum Cylinder Thickness for Handling</u>

Pipe Diameter	= 54 inches
Minimum Cylinder thickness Per AWWA M11	= Based on D/t ratio of 240 = 0.225 inch
Min. Cylinder thickness Per SDCWA Requirement	= 0.50 inch

<u>Internal Pressure</u>

Design Pressure	= Maximum operating pressure plus 50 psi, varies from 250 up to 525 psi
Steel Grade Considered	= Grades 36 and 40 with 60,000 psi tensile strength
Design Circumferential Stress	= 50% of minimum yield strength = 18,000 or 20,000 psi
Maximum Surge Pressure	= 1.33 times the design pressure.

<u>External Earth and Live loads</u>

Minimum Earth Cover	= 8 ft.
Average Earth Cover	= 12 ft.
Maximum Earth Cover	= 20 ft. (95% of the pipeline has a maximum cover of 16 feet)
Unit Weight of Soil	= 120 lb/ft^3.

Earth Load	= Prism load
Live Load	= AASHTO HS20 (M11 Table 6-3)
Modulus of Soil Reaction, E′ Per Geotechnical Report	= Embedment material varies between 2200 to 2500 psi
Maximum Pipe Deflection	= 1 % (M11 allows 3% for mortar-lined and flexible (coated)
Height of Ground Water Above Pipe	= 1 foot (For external buckling calculations)

Field Welded Joint Analysis

The pipeline's joint analysis included checking the longitudinal stresses due to PA or longitudinal stresses due to Poisson's effect and temperature differential stresses per the SDCWA's design guidelines. As requested by the City of Carlsbad's consultant, the joint analysis was performed per ASCE Manual No. 79 for Steel Penstocks which utilizes the ASME joint efficiency factors for butt and lap-welded joints to account for the joint strength and eccentricity. Figure 2 shows field welding of the interior of a double lap-welded joint.

The following design criteria and assumptions were made:

- The steel considered for the project was either grade 36 or 40 with a minimum tensile strength of 60,000 psi for both grades.
- The design pressure, which is the operating pressure plus 50 psi, was used for determining the shell thickness. The maximum allowable circumferential stress for the design pressure was 50% of the minimum yield strength.
- For joint analysis the operating pressure was used since the ASCE Manual 79 design basis is the normal operating pressure.
- The allowable primary stress for joint analysis will be 1/3 of the minimum tensile or 2/3 of yield strength in conformance with the ASCE Manual. For this project the 1/3 of the tensile strength will be controlling, which is 20,000 psi. Therefore, the primary design stress will be 20,000 psi.
- Although the use of a stress increase factor of 1.5 for temperature differential Δ, which is considered a secondary stress per ASME, and Poisson's primary stresses are justified, a factor of 1.25 was used based on the recommendation of the City of Carlsbad's consultant.
- The Δ temperature is 30 degrees Fahrenheit.
- Only Δ temperature and Poisson stresses are combined.
- The joint efficiency factors for the single lap and double lap-welded joints are 0.45 and 0.55, respectively in conformance with the ASCE Manual (Table 3-3).
- Although single lap-welded joints in the lower pressure thrust restraint areas could be justified, all joints in thrust restraint areas due to PA will be double lap-welded.
- The minimum joint lap for the lap-welded joints was 4.0 inches which was specified.

A summary of allowable longitudinal stresses for the different loading conditions is presented in the table below:

A	B	C	D	E
PA*, psi	Poisson's effect*, psi	Temp.& Poisson with 1.25 stress factor, psi	Poisson &Temp. for single lap-welded joint using weld joint reduction factor of 0.45, psi	Poisson &Temp. for double lap-welded joint using weld joint reduction factor of 0.55, psi
20,000	**20,000**	**25,000**	**11,250**	**13,750**

*Allowable stress for single lap-welded joints in thrust areas or Poisson's effect will be 20,000 x 0.45 = 9,000 psi and for double lap-welded joints will be 20,000 x 0.55 = 11,000 psi.

Figure 2. Field welding of interior double lap-welded joint.

Summary of Results

The required wall thicknesses for pipe classes 250 up to 525 psi varied between 0.50 inch and 0.725 inch using grades 36 or 40 steel. Pipe Classes 250, 300, and 350, which are 25% of the pipeline's total length, had 0.50 inch wall thickness since the minimum 0.50 inch wall thickness controlled the design. The wall thicknesses for Class 400, 450, 500 and 525 were 0.5625, 0.625, 0.6875 and 0.725, respectively.

The cylinder thicknesses based on hydrostatic pressure designs were checked for the external loads and allowable external buckling pressures. All external load design check results, including the sensitivity analysis of the native soil stiffness to pipe embedment stiffness, satisfied the design conditions and criteria.

Based on the joint analysis results single lap-welded joints were justified for pipe classes up to class 450 and double lap-welded joints for pipe classes 500 and 525. For conservatism, single lap-welded joints were recommended for pipe classes up to class 400 only and double lap-welded joints for higher classes.

FINAL CARLSBAD CONVEYANCE PIPELINE DESIGNS

Hydrostatic Pressure Design for the Shell Thickness

The final designs were based on the revised design pressure and surge pressure hydraulic profiles and clarification addendum for SDCWA's Design Guide. Since the final hydraulic analysis was based on utilizing a surge tank, the design pressure was based on the actual operating pressure plus 25 psi only in lieu of the 50 psi assumed in the preliminary designs. The surge pressures were 18 to 23% higher than the design pressures and 25 to 30% higher than the actual operating pressures. The allowable design stresses for the design pressure and surge pressure were 18,000 psi and 21,000 psi, respectively. Thirteen wall thicknesses and pipe classes from 0.50 inch up to 0.866 inch were established; a very short length in a tunnel had 0.895 inch wall thickness. Von Mise's Equivalent stresses for circumferential and longitudinal stresses were also checked during the design analysis. Seismic analysis was also performed for the 2,475 year return earthquake for the project site using Finite Element modeling and non-linear stress-strain relationship.

Design Conditions and Criteria

Pipe diameter	= 54 inch
Cylinder ID	= 55 inch
Design Pressure	= Maximum operating pressure plus 25 psi, varies from 250 up to 337 psi
Surge pressure	= 335 up to 662 psi
Min. steel yield strength	= 40, 000 or 42, 000 psi
Min. tensile strength	= 60,000 psi
Design hoop stress at design pressure	= 18,000 psi
Design hoop stress at surge pressure	= 21,000 psi
Allowable seismic stresses or strains for applicable load combinations	= 0.15%

Field Welded Joint Analysis

The joint analysis procedure for the lap-welded joint consisted of calculating: (1) longitudinal stresses due to PA or (Poisson's effect + Δ temperature); (2) bending stress due to lap-joint eccentricity; and (3) the collar effect of the bell. These stresses were added and compared to the allowable stresses for both operating (design) and surge conditions. Seismic analysis for operating conditions was also performed.

Although single lap-welded joints could be justified for the lower pressures, double lap-welded joints were utilized for the entire project. The minimum joint lap required was 4 inches, however a more conservative value of 4.5 inches was used based on 5 times cylinder thickness, t, for the thickest cylinder.

Design conditions and criteria for the double lap-welded joints are:

Allowable longitudinal stress PA or (Poisson's effect +Δ temperature)	= 50% of yield strength
Allowable longitudinal stress PA or (Poisson's effect +Δ temp) +bending, operating conditions	= 75% of yield strength
Allowable longitudinal stress PA or (Poisson's effect +Δ temp.) +bending, surge conditions	= 80% of yield strength
Allowable longitudinal stress PA or	

(Poisson's effect +Δ temp.) +bending
+ collar effect, operating condition = 85% of yield strength
Allowable longitudinal stress PA or
(Poisson +Δ temp.) +bending+ collar
effect, surge condition = 90% of yield strength
Von Mise's equivalent stress
at operating condition = 85% of yield strength
Von Mise's equivalent stress
at surge condition = 90% of yield strength
Allowable seismic stresses or strains
for applicable load combinations = 0.25% strain

Analysis Procedure for Bending due to Joint Eccentricity

The procedure for checking the bending moment and shear forces across the lap joint is based on beams on an elastic foundation analysis as outlined in Roark's Formulas for stress and strain, fifth edition Table 30, case 16. The moments at the bell and spigot ends will be superimposed by the effect of the moment at the opposite ends; the moments reduce exponentially. The double-lap-welded joint and location of the maximum net moment at the toe of the spigot at X1 = (ℓ + t) and bell at X2 = (ℓ + t) weldments are depicted in Figure 3. The dampening effect of the $M_o/2$ moment at the toe of weld at the other end is a function of the lap joint width ℓ; therefore longer lap length will result in lower net moment. The maximum net moment at the toe of the weldments will be equal to $M_o/4$ plus the dampened effect of $M_o/2$ from the other end, where M_o = T (thrust due to PA) x (t + 0.06 inch gap).

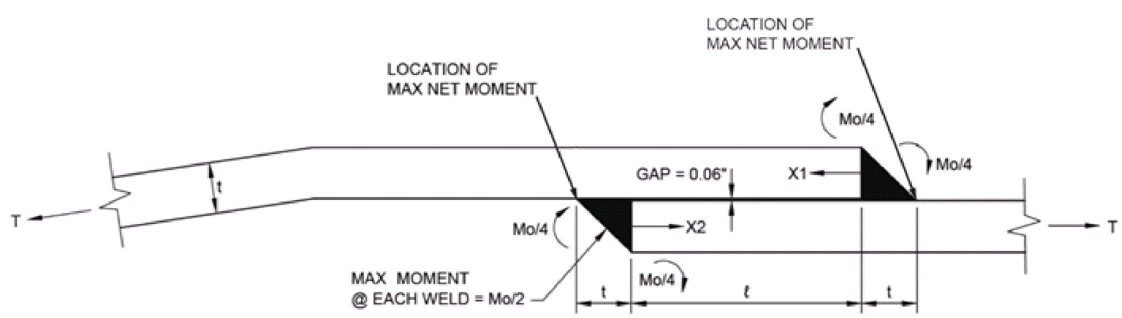

Figure 3. Double-Lap Welded Joint Detail and Acting Bending Moments.

Collar Effect Check

Roark's case 13 from the fourth edition for a cylindrical vessel with reinforcing rings of cross section A was used to estimate the collar effect and calculate the rim bending stresses and also the shear at the weldments.

This is an approximate analysis only since the bell is not a solid ring, and it is attached to the main cylinder through the two weldments; the bell is also subject to some rotation. Therefore, only 50% of the calculated ring bending stress was assumed to be effective.

Summary of Results

The 13 wall thicknesses established for the pipeline were based on the hydrostatic pressure design requirements and varied between 0.50 and 0.866 inch. The 0.50 inch thickness was required for 20% of the total length of the pipeline; another 5% of the total pipeline length required 0.512 inch thickness. The double lap-welded joint analysis results confirmed that all the project design criteria were met. The joint analysis results were also verified by the finite element modeling results.

Figure 4 shows a 54-inch diameter pipe section being lowered in a trench box with its bell end being prepared for insertion over the spigot end of an already installed pipe section.

Figure 4. Lap Joint Assembly. The pipe section on the left hand side is being positioned opposite the already installed pipe section for lap joint assembly and welding.

COMPARISON BETWEEN THE PRELIMINARY AND FINAL DESIGN RESULTS

For about 20% of the total project length, the wall thicknesses of the preliminary designs and final designs did not change since the 0.50 inch wall thickness controlled the designs; the final designs for another 5% of the total length required 0.512 inch thickness. The final design wall thicknesses for about 75% of the total length of the pipe increased by an average of 10% for the following reasons:

- The static head has increased by 85 feet due to the change of the final delivery point.
- The allowable stress of 21,000 psi at surge pressure controlled the designs.

Both the preliminary and final welded joint analyses justified the use of the double lap-welded joints for the entire pipeline.

STEEL CYLINDER FABRICATION REQUIREMENTS

The Carlsbad Conveyance Pipeline was manufactured in conformance with SDCWA's Standard Specifications which are more stringent than the AWWA C200 requirements. Some of these requirements include tighter tolerances for the cylinder outside diameter, allowable spiral and straight weld seam offsets, and weld seam repairs. Due to the critical application of this project 100% radiographic testing was required for all weld seams of the pipelines. Also the specification required that the yield strength of adjoining pipe sections shall not exceed 5 ksi.

Steel Cylinder Manufacturing Processes

The fabrication of the steel cylinders with 0.50 inch up to 0.90 inch wall thicknesses for the Carlsbad Conveyance Pipeline required two manufacturing processes: (1) spiral-seam pipe made from coiled steel by a continuous forming and welding process for thicknesses up to 0.70 inch; and (2) straight-seam pipe made from plate steel by rolling and welding the longitudinal seam of 10-feet long segments and assembling four segments with girth seam welding for thicknesses over 0.70 inch. The steel cylinders, with seven thicknesses varying from 0.50 inch up to 0.70 inch, for 64% of the total length of the pipeline were fabricated by the spiral forming and welding process. The remainder of the cylinders, with six thicknesses varying from 0.72 inch up to 0.90 inch, were fabricated by the plate rolling and automatic welding the straight and girth seams process.

The thickness limit of steel cylinders fabricated by the spiral process is usually established by many factors which include: (1) D/t ratio, since it takes more power to form cylinders with a low D/t ratio; (2) yield strength of the steel, since it is takes more power to form higher yield steel; (3) coil width, since it takes more power to form wider coils; (4) allowable weld seam offsets; (5) forming capacity of the spiral welding machine; and (6) availability of thicker coil steel.

Steel Cylinder Material

The steel coil material is in conformance with ASTM A1018-SS grade 40 for the 0.50 inch wall thickness and grade 40 modified to minimum yield strength of 42,000 psi for wall thicknesses greater than 0.50 inch; the minimum tensile strength of both grades was modified to a minimum 60,000 psi. Plate steel material is in conformance with ASTM A36 modified to a minimum yield strength of 42,000 psi or ASTM A572 grade 42. The steel coils and plates are fine grained, fully killed, and manufactured using a continuous casting method. Additional requirements for the steel coils and plates included a maximum carbon equivalent of 0.45 per the AWS formula and a minimum 25 ft-lb Charpy V-Notch impact toughness at a temperature of 30 degrees Fahrenheit.

SUMMARY AND CONCLUSIONS

The 10 mile long, 54-inch diameter Carlsbad Conveyance Pipeline, from the pump station of the desalination plant in Carlsbad, California to the SDCWA's Twin Oaks Treatment Plant, is a critical component of a high profile desalination project. The 54 MGD desalination project, which will be completed in 2016, will be the largest ocean water desalination project in the Western Hemisphere.

Both the preliminary and final designs of the 54-inch diameter steel pipeline for a maximum design and surge pressures of 337 psi and 662 psi, respectively, were based on conservative design criteria for the pipe shell and lap-welded joints. The joint analysis for the preliminary designs was based on ASME's joint efficiency factors and allowable stresses, while the joint analysis of the final designs was based on limiting the joint stresses, due to all forces including bending from the lap-joint eccentricity, to a threshold below yield except seismic load.

The designs for almost 25% of the total length of the pipeline based on the preliminary and the final designs were identical since the designs were controlled by the minimum 0.50 inch cylinder thickness requirement. The required steel cylinder wall thicknesses for the remainder of the pipeline length, based on the final designs, were about 10% greater than the thicknesses based on the preliminary designs for two reasons: (1) the static head of the final designs was 85 feet greater than the static head of the preliminary designs; and (2) the allowable circumferential stress at surge pressure was limited to 21,000 psi due to the critical system operating conditions.

The Carlsbad Conveyance Pipeline was manufactured with stringent fabrication tolerances and non-destructive testing requirements due to the critical application of this project. The fabrication of the steel pipeline, with cylinder thicknesses varying from 0.50 inch up to 0.90 inch, required two manufacturing processes based on cylinder thicknesses.

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Analysis of a Steel Pipeline in a Seismically Active Region

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Abstract: The Carlsbad Desalination Project uses a pipeline to deliver water from a seawater desalination plant located near sea level to the San Diego County Water Authority's Second Aqueduct connection facility located at an elevation of about 560 ft above sea level. The approximately 10-mile long steel pipeline is 54 inches in diameter, has a wall thickness varying from 0.500 to 0.895 in., and is joined with double lap welds. Along the pipeline alignment there are simple bends, compound bends, and tees. In addition, certain parts of the pipeline are installed within a steel casing in a tunnel. The authors analyzed the stress states of the pipe walls and of the double lap welded joints, assessed the thrust restraint behavior of the pipe, and analyzed the stresses and strains in areas where the pipeline is shielded by tunnel casing and incorporates a tee connection. These analyses accounted for the effects of combined loads resulting from internal working and transient pressures, temperature change, and seismic longitudinal strains and curvatures of the soil on the pipeline. This paper discusses the analytical procedures used in the stress analyses, including material nonlinearity and pipe-soil interaction, for pressure pipelines subjected to seismic wave propagation effects.

INTRODUCTION

The Carlsbad Desalination Project incorporates a conveyance pipeline that is approximately 10 miles long and delivers water from a seawater desalination plant located near sea level to the San Diego County Water Authority's Second Aqueduct connection facility located at an elevation of about 560 ft above sea level. The 54-inch inside diameter pipeline is divided into eastern and western branches, has wall thicknesses that vary from 0.500 to 0.895 in., and uses double lap welds to join its segments (Bid Set Drawings, 2011). Along the pipeline alignment there are simple bends, compound bends, and tees, as well as portions that are installed within a 72 in. diameter steel casing in a tunnel.

The purpose of this paper is to show how finite element analyses can be effectively employed in the design process to produce economical and safe designs for pipelines subjected to the combined effects of internal pressure, temperature differential between the construction and operation conditions, and seismic strains imparted to the pipeline from seismic wave propagation in the soil. The scope of this paper is limited to the analysis of stresses and strains in the wall of the steel pipes at the joints and away from the joints for straight length of pipe, for areas near bends and tees, and for areas within a steel casing in a tunnel subjected to the design load combinations.

To investigate the behavior of the pipeline, we developed finite element models (FEMs) to capture the pipeline performance at different locations with pipe wall thicknesses, maximum pressures, and seismic loads that vary along the pipeline. These FEMs simulated the exact geometry of the pipe and joints and the nonlinear material behavior of the steel by accounting for its post-yield behavior. We used these models to assess the effect of pressure and temperature loads, coupled with the seismic ground motion strains for both tension and compression waves transmitted from the soil into the pipeline.

The loads acting on the pipeline are internal pressures, thermal, and seismic. Three different design pressures are defined along the pipeline: the maximum operating pressure ranging from 256 psi to 513 psi, the design pressure defined as the operating pressure plus 25 psi, and the maximum surge pressure ranging from 335 psi to 661 psi. Thermal loading is expressed as a differential temperature of $\pm 30^{o}$F between the construction and operation conditions. The seismic soils strains are based on wave propagation effects of an MCE earthquake, which has a return period of 2,475 years.

The design criteria against which the FEM analysis results are evaluated require the following (SDCWA Design Manual, 2007, and Tetra Tech, 2012):

- Circumferential stress at design pressure shall be below 18 ksi.
- Circumferential and longitudinal stress at surge pressure shall be below 21 ksi, which is 50% of the material's yield strength.

- Combined longitudinal stress from axial force and bending moment due to design pressure and surge pressure, both with thermal loading, shall be below 75% and 85% of yield, respectively.
- von Mises stresses due to design pressure and surge pressure, both with thermal loading, shall be below 85% and 95% of yield, respectively.
- Longitudinal strains from operating pressure, thermal loading, and seismic loading shall be below 0.2% (minor yielding is permitted).

Using the USGS data for MCE earthquake along the pipeline, we determined the peak ground acceleration (PGA) and the mapped spectral accelerations for short periods (S_s) and for 1 sec period (S_1) and then calculated the peak ground velocity (PGV) using the procedure presented by Seed and Idriss (1982). The ground strain is then the ratio of the PGV to the apparent wave velocity, C. The apparent wave velocity was selected by the geotechnical engineer based on a literature search of recorded earthquake data during previous historic earthquake events; a value of 6,560 fps was selected (Leighton Consulting, 2013). This resulted in soil strains of 374 and 324 microstrains for the western and eastern branches, respectively. The effects of changes in curvature of soil from seismic wave propagation are very small and only increase the peak ground strains to 375 and 325 microstrains. The USGS data and resulting PGV and ground strain values are summarized below in Table 1.

Table 1 – Peak ground and spectral accelerations, peak ground velocities, and the resulting ground strains used in the finite element analyses

	PGA	S_s	S_1	PGV	Ground Strain
Eastern branch	0.490 g	1.176 g	0.445 g	25.50 in./sec	325 $\mu\varepsilon$
Western branch	0.566 g	1.337 g	0.504 g	29.46 in./sec	375 $\mu\varepsilon$

ANALYSIS OF PIPE AND JOINTS IN STRAIGHT LENGTHS OF PIPELINE

Description of Models

In straight lengths of pipeline, the soil and the pipe move together; therefore, soil strain is transmitted directly to the pipe and its joints. The geometry of the joint gives rise to bending stresses in the pipe wall and stresses at the welds. To evaluate these stresses, we developed FEMs that represent a 1 in. arc width of the steel pipe in the vicinity of the welded joints and extend 44 in. along the spigot pipe and 57.5 in. along the bell. Figure 1 shows a geometric representation of the pipe joint region, with the darkly shaded portion representing the extent of the FEMs. A total of 13 FEMs were generated and analyzed, one for each of the pipe wall thicknesses used along the 10-mile pipeline.

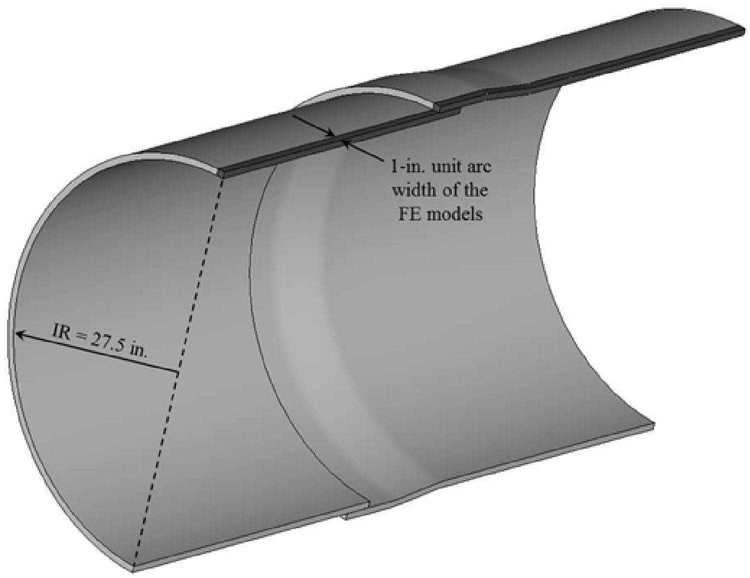

Figure 1 – Solid Geometry Representation of Pipe Joint Region

Each FEM consists of a strip of plate elements matching the exact geometry of the pipe in the radial direction. The bell and spigot portions of the FEMs are joined with plate elements that represent the fillet welds placed at the ends of the pipe segments. Figure 2 shows a side view of the FEMs, and a close-up view of the area at the joint, with localized bell region dimensions, is shown below in Figure 3.

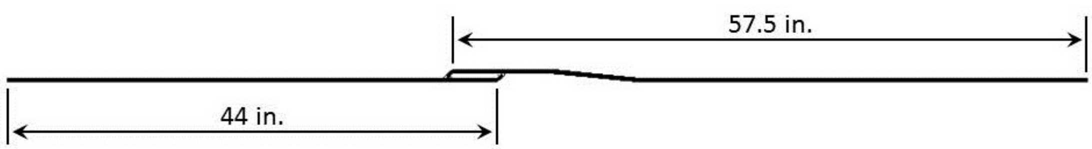

Figure 2 – Side View of the FEM

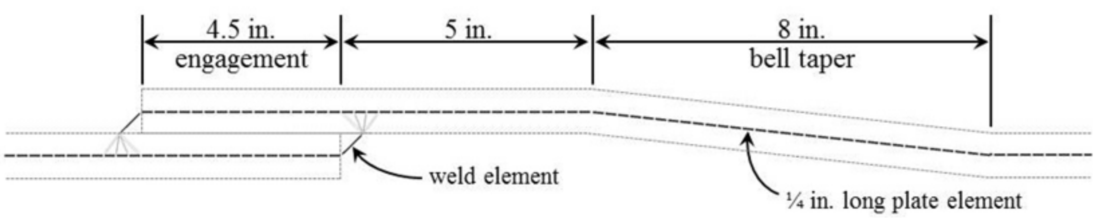

Figure 3 – Detail View of Joint Region Geometry. (The outline of pipe wall is shown in gray.)

The boundary conditions consist of axial symmetry along the longitudinal edges to allow the pipe to move freely in the radial and longitudinal directions (R and Z), with rotation permitted about the circumferential axis of the pipe (θ), while restraining displacement in the circumferential direction and rotations about the R and Z axes. The ends of the model are restrained against longitudinal displacement for all loads except for seismic load, where imposed displacements are used to simulate the ground strain.

To accurately capture the material behavior, we used a trilinear material model having three distinct behavior zones, as shown below in Figure 4: 1) an elastic zone, extending to an initial yield point of 42 ksi at 0.145% strain, 2) a plastic zone with no appreciable rise in stress extending to 42.1 ksi at 2.0% strain, and 3) a strain hardening zone up to a stress of 60 ksi at 20% strain before rupturing thereafter.

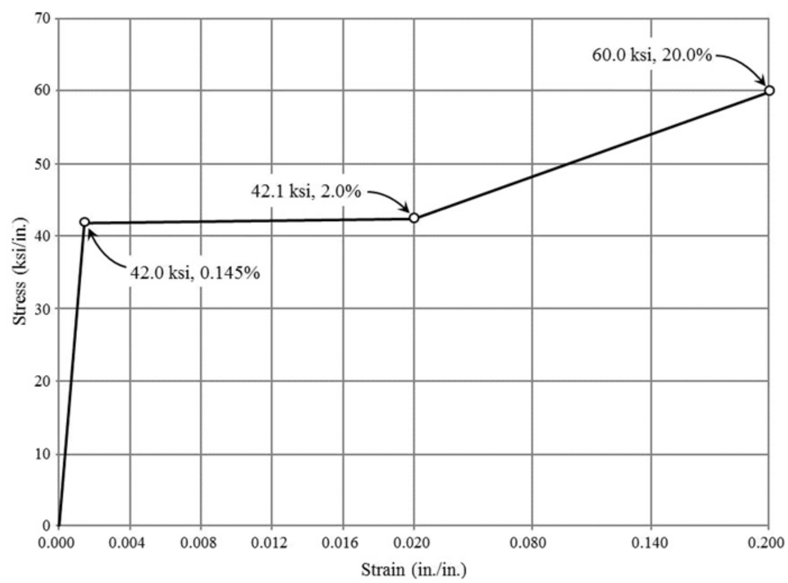

Figure 4 – Nonlinear Stress-Strain Relationship Used in FE Models

Internal pressure is applied as a uniform pressure on the inside surface of the pipe. The imposed axisymmetric boundary conditions capture the Poisson effect in the longitudinal direction resulting from circumferential strains. The pressures used in the analysis consist of the operating pressure, the design pressure defined as the operating pressure plus 25 psi to account for the increased pressure during pump startup and shutdown, and surge pressure defined as the envelope of the maximum pressures resulting from different scenarios that can produce a transient surge event.

The temperature differential of $\pm 30^{o}$F is applied directly since the material model incorporates steel's thermal expansion coefficient. The seismic ground strains of 375 and 325 mircrostrain are imposed as longitudinal displacements at one end of each FEM while the opposing end is restrained.

Results from FEMs

The results from the FEMs of the straight lengths of pipeline showed that all longitudinal stresses were well below the 21 ksi stress limit, and the combined stresses and von Mises stresses were also well within the stated design limits. In some cases the surge pressure-induced maximum circumferential stress exceeded the specified 50% of yield stress limit, reaching 23.7 ksi, i.e., 56% of yield; however, this does not result in risk of failure. The argument was accepted resulting in no change in pipe thickness.

Under the combined effects of operating pressure, temperature change, and seismic loading from the design seismic event, the combination of membrane forces and local bending of the pipe wall near the joints will not cause yielding. The maximum stress on the surface of the pipe occurs in the bell at the internal weld location and results in a strain of 0.139%, which is below the 0.145% yield strain of the steel. Figure 5 shows the deformed shape of the model, at 100x amplification, in the vicinity of the joint along with a contour of the associated strains. For the compression wave acting on the pipe, regardless of the presence of internal pressure, the stresses and strains are well below the material yield point.

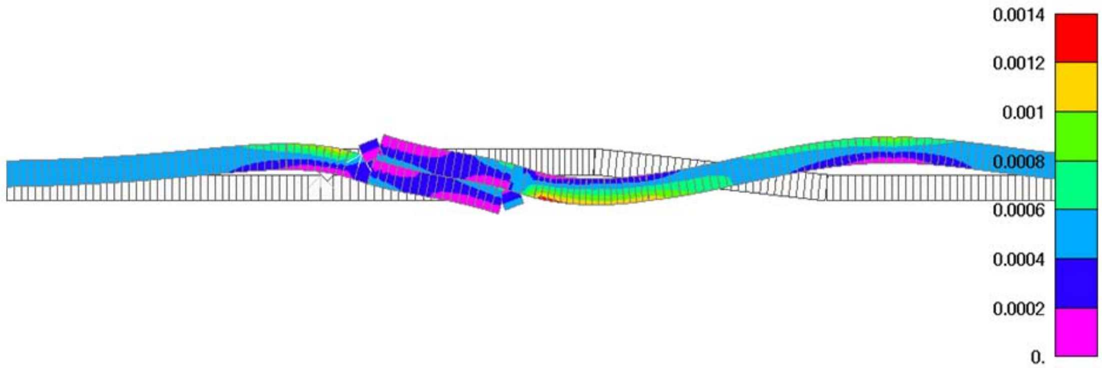

Figure 5 – Deformed Shape at 100x Amplification with Strain Contour

ANALYSIS OF THRUST RESTRAINT

To determine the response at bends due to seismic ground strain, temperature load, and pressure-induced thrust, we developed FEMs that each represent a longitudinal portion of the pipeline having a single or multiple bends. In this paper we present the case of a single 45-degree elbow for a 0.500 in. thick pipe wall with an operating pressure of 255 psi. This FEM captures the actual geometry of the pipeline at the bend along with pipe-soil interaction.

Description of Model

The pipeline is modeled with beam elements, and the soil is modeled with distributed nonlinear springs (pipe-soil-interaction, PSI, elements) that simulate the stiffness of the soil against lateral pipe displacement and the pipe-to-soil friction with a friction coefficient of 0.4. We used a conservative lateral soil stiffness of 3,400 lbs/in./in., corresponding to a coarse-grained backfill, dense and medium dense. The friction force resists longitudinal displacement of the pipe, which is important because the tensile seismic wave propagation and pressure induced thrust near the elbow results in friction forces in opposite directions and can result in changes in the direction of friction force along the length of the pipeline. (Note that changes in the direction of friction force cannot be easily captured by simple analytical procedures that lend themselves to hand calculations.)

Seismic strain was applied using a pseudo-thermal load, with the temperature change calculated by $\Delta T_{EQ} = -\varepsilon_{EQ}/\alpha$ and tensile seismic strain taken as positive. This temperature change that represents seismic strain is applied to the pipe while the nodes on the soil-side of the PSI elements are held fixed; in this way the soil resistance against longitudinal displacement of the pipe is simulated by the frictional resistance of the PSI elements. The specified temperature differential was added to the pseudo-thermal load of the seismic strain. The thrust resulting from internal operating pressure was applied as a concentrated force at the bend location for cases with a tensile seismic wave; however, the thrust force is not included with a compressive seismic wave, because pressure induced thrust reduces the compressive stress in the pipe wall.

Results from FEM

The resulting maximum longitudinal stress for the operating pressure plus negative temperature change plus seismic tension wave loading was 30.9 ksi. The analysis was only performed for a seismic tension wave because it has already been shown that tension waves control the maximum stress at the bends in all cases. Figure 6 shows the results from the FEM. The bending stresses, as expected, peak in the vicinity of the elbow; however, the axial stresses actually decrease as one approaches the elbow. Away from the elbow, the maximum stress corresponds to the maximum seismic soil strain, as evidenced by the flat portions of the axial stress curve at either side of Figure 6. The seismically-induced axial stresses then decrease near the elbow because the elbow can move into the soil, thereby reducing the net strain over the pipe's cross-section, though this reduction is more than offset by the corresponding stress increase due to bending that arises at the bend.

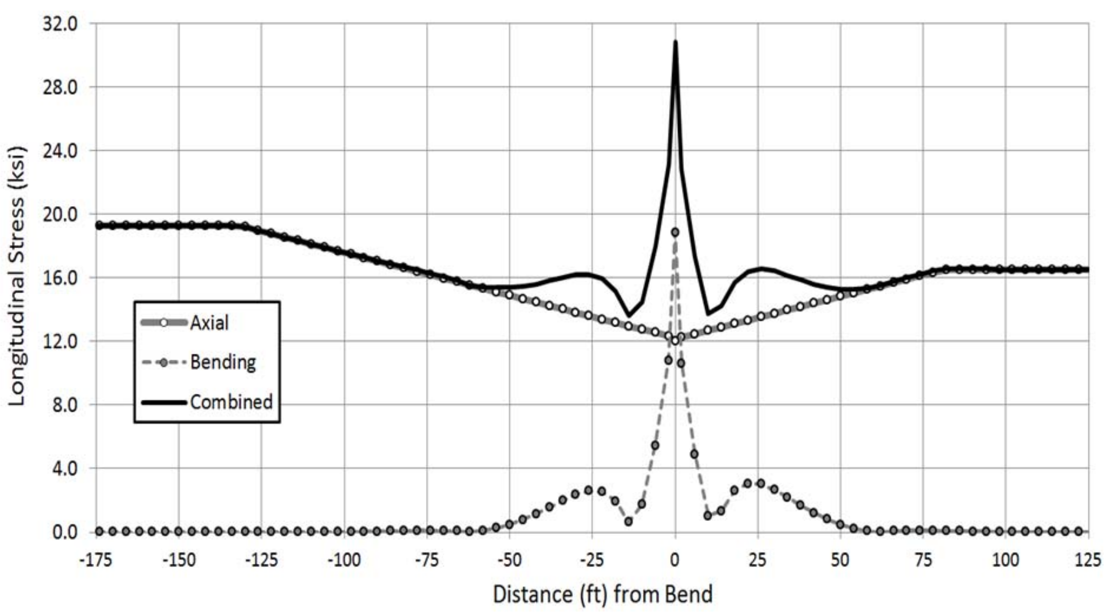

Figure 6 – Axial, Bending, and Combined Longitudinal Stresses for 45-Degree Bend Subjected to Operating Pressure, -30°F Temperature Change, and Tensile Seismic Ground Strain

This analysis does not account for higher localized stresses at the joints and stress intensification at the elbow miters; however, it indicates that stresses are relieved near the sharp bends as the pipe moves into the soil, resulting in longitudinal stresses that are significantly less than the yield strength of the pipe with no plastic strain expected.

Using the peak stresses in the pipe wall of the FEM, we then examined the corresponding stresses that would occur in a joint located anywhere on the straight legs of the bend. To do this, we reanalyzed the strip model of the pipe and joint for a 0.500 in. wall thickness by assuming that the joint was located at the worst possible location and then applying a uniform (conservatively assumed axisymmetric) tension force resulting from the peak combined longitudinal stress. The results from the strip model reveal a maximum von Mises stress that exceeds 42 ksi, indicating that the pipe wall experiences varying degrees of yielding. The associated peak strain in the longitudinal direction is 0.53% and exceeds the 0.20% strain limit criterion.

Because the above results indicated strains exceeding 0.2%, we refined the thrust analysis model to account for the 2.5 D bend radius. The results from the refined model indicate that the 2.5 D bend radius substantially reduces the demands in the pipe wall, with the maximum combined stress reducing from 30.9 ksi, as shown above in Figure 6, down to 23.3 ksi, as shown below in Figure 7.

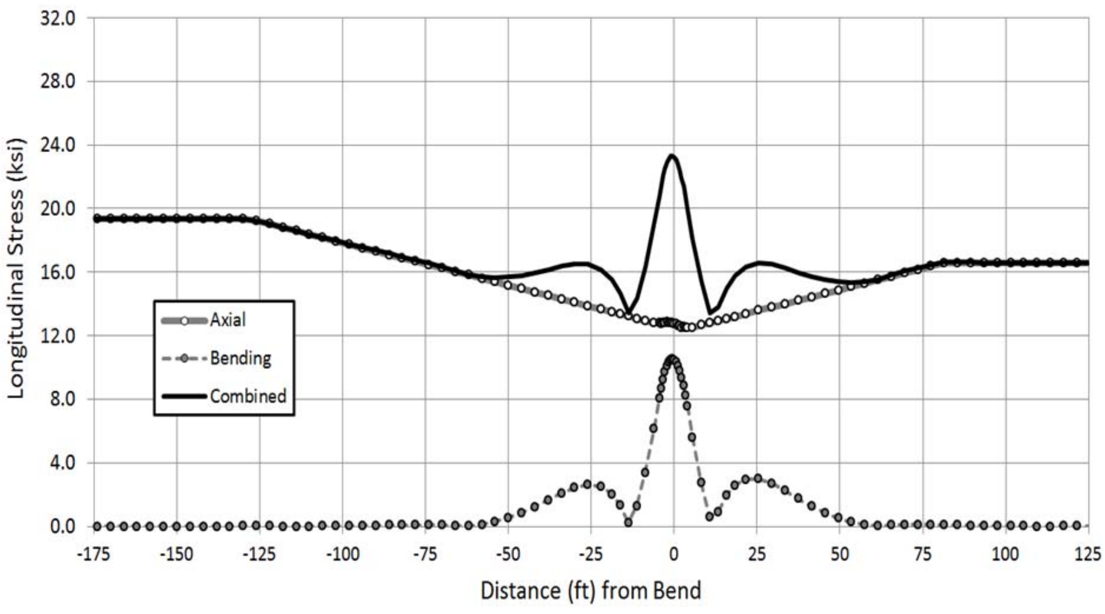

Figure 7 – Axial, Bending, and Combined Longitudinal Stresses for 45-Degree Bend Subjected to Operating Pressure, -30°F Temperature Change, and Tensile Seismic Ground Strain after Accounting for the 2.5 D Bend Radius

We recomputed the strains near the joint using the strip model and found the maximum strain to be 0.13%, which is well below the 0.20% criterion.

ANALYSIS OF PIPELINE WITHIN STEEL CASING IN TUNNEL

Description of Model

Near its western end, the pipeline passes through a tunnel and into a tee connection to turn upwards within a 43-foot high riser. At this location, the pipe wall thickness is 0.866 in., and the riser wall thickness is 0.895 in. To analyze this location for seismic loading conditions we generated a FEM to simulate the tunnel portion of the pipeline, including the lower half of the riser, its tee connection, and the crotch plates that stiffen the tee. An isometric view of the region near the tee is shown below in Figure 8, with the start of the bar element portion of the model visible at the left side of the image.

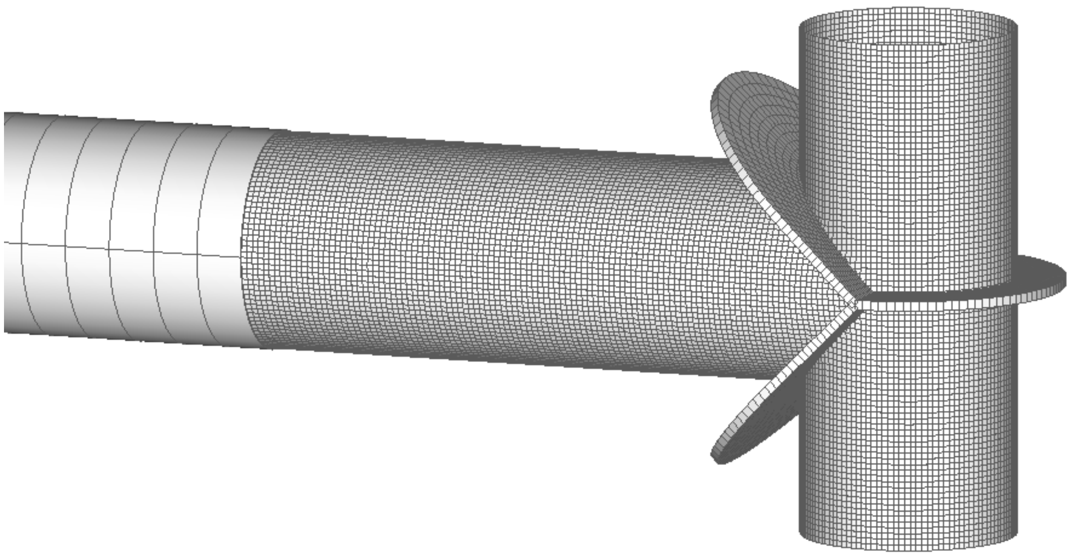

Figure 8 – FEM of Tee Region of the Pipeline

Each FEM used bar elements for the areas away from the riser and tee but captured the localized stresses at and near the tee with a three-dimensional mesh of plate elements as shown in Figure 8. The meshed portions of the FEMs utilized the same non-linear material property as presented above in Figure 4. For areas away from the tee and riser, where yielding is not expected to occur, a linear-elastic material was used.

The pipeline was supported on a series of soil springs. For the lateral soil stiffness along the riser, we used a 7,000 lbf/in./in. stiffness, which corresponds to a very dense coarse-grained soil. For the vertical soil stiffness, we assumed that the surrounding rock for the tunnel portion is effectively rigid and computed the stiffness of the CLSM material around the pipe, yielding a 12,000 lbf/in./in. stiffness. The frictional resistance in this area is minimal, resulting from only the weight of the pipe and the CLSM material over the pipe. For the near-surface portion, we used the same stiffness and frictional resistance as established in the thrust restraint analyses. The spring supports are shown schematically in Figure 9.

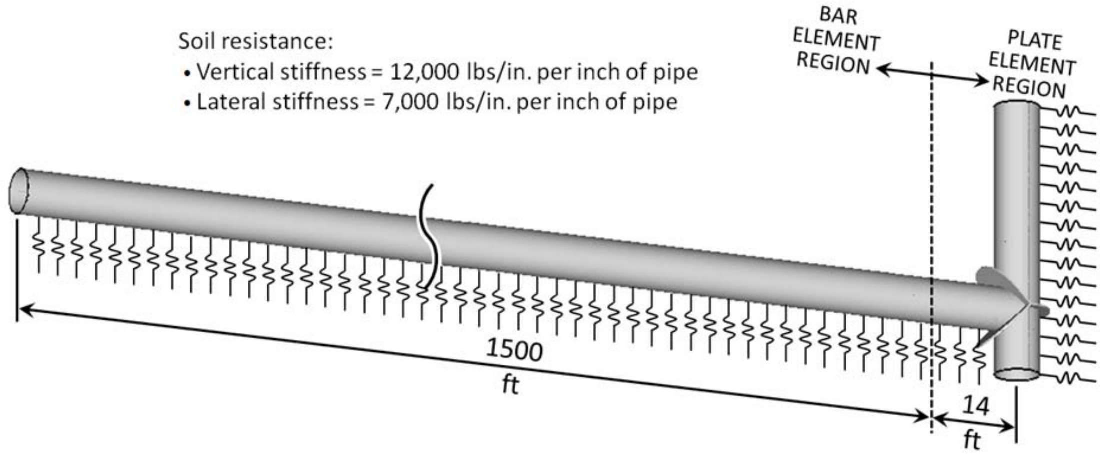

Figure 9– Spring Supports along the FEM

We applied the operating pressure, 517 psi, to the interior pipe surfaces of the meshed region of the FEM, with the Poisson effect directly accounted for due to the behavior of the plate elements in the circular cross-section of the pipe. For the bar element region, we simulated the Poisson effect by calculating and imposing a temperature load, specifically a temperature decrease, that produces the equivalent axial shortening from the Poisson effect. We imposed the $\pm 30^{o}$ F temperature on all pipe, riser, and crotch plate elements, with the variable sign indicating that the effect of temperature change may be additive or offsetting depending upon the loading that is being considered. To impose the 375 microstrain seismic load, we applied the same pseudo-thermal load as discussed above in the thrust restraint portion of this paper.

Based on our prior analyses, we recognized that the presence of internal pressure can offset the effects of other loads. Because of this, we analyzed the FEM 1) for a compressive seismic wave coupled with a temperature change of $+30^{o}$ F both with and without internal operating pressure and 2) for a tensile seismic wave coupled with a temperature change of -30^{o} F both with and without internal operating pressure.

Results from FEM

In the pipe wall, the maximum von Mises stress occurs for the compressive seismic loading when accompanied by the 517 psi internal operating pressure and the $+30^{o}$ F temperature increase and is equal to 42 ksi as shown in Figure 10, indicating yielding. This occurs at the extreme fiber on the inside surface of the pipe at the point where the crotch plates are joined to the pipes; it is accompanied by a total strain of 0.233% that is limited to a small area, as shown below in Figure 11. The von Mises stresses in the crotch plates did not exceed 27.6 ksi, or 77% of the plates' 36 ksi yield strength.

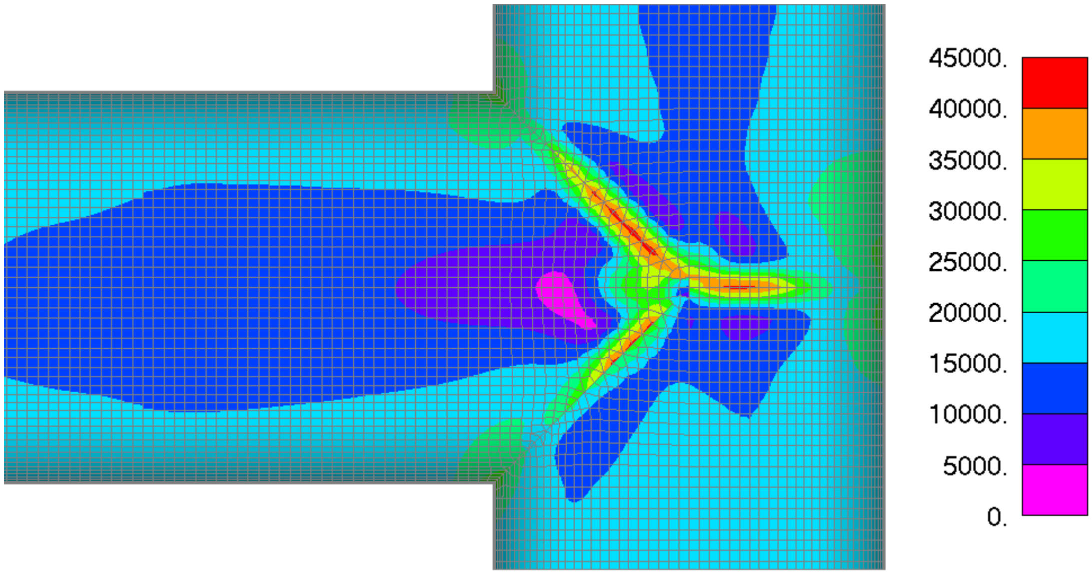

Figure 10 – Plot of von Mises Stress on Pipe Inside Surface for Seismic Compression plus Operating Pressure plus Temperature Increase

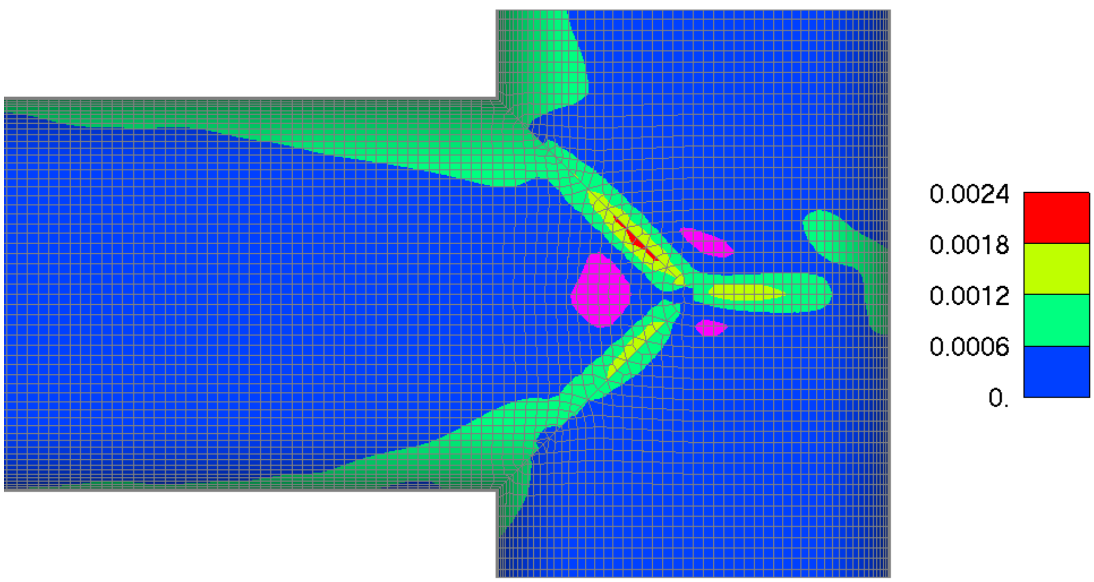

Figure 11 – Plot of Strain on Pipe Inside Surface for Seismic Compression plus Operating Pressure plus Temperature Increase

Using the raw elemental force output from the FEM, we also computed the peak membrane forces and stresses in the elements for each load combination. This resulted in a maximum membrane stress (no bending) of 30.5 ksi in the pipe wall, which corresponds to 73% of the 42 ksi yield stress. We performed a similar calculation for the crotch plates and found the peak membrane stress to be 28.7 ksi, or 80% of the 36 ksi yield stress of the crotch plates. Based on these results, the design was found to be acceptable.

CONCLUSIONS

Design of steel pipelines subjected to the effects of internal pressure, temperature, and earthquake in seismically actively regions can be performed effectively, safely, and economically using finite-element models (FEMs) that faithfully and accurately represent the geometry of the pipeline alignment, including the joints and fittings, and that account for material nonlinearity and soil-structure interaction. For the Carlsbad pipeline, the paper presents the general analysis procedure and the results for the joints in a straight length of pipeline (for which an axially symmetric FEM was developed), pipes near single and multiple bends (for which a pipe-soil interaction FEM was developed), and a riser with a tee connection to a pipeline within a tunnel (for which a 3D FEM with pipe-soil interaction was developed).

The results of these analyses show the following:

1. For the straight length of pipeline subjected to the combined effects of internal pressure, temperature decrease, and tensile strain in the soil from seismic wave propagation, the FEM results show that the maximum stress in the joint will not cause yielding. The maximum stress in the extreme fiber occurs in the spigot and produces a strain below the 0.145% yield strain and well below the 0.2% strain limit in the design criteria. Without the seismic loading, the maximum stress is below 75% of yield, and the von Mises stress in the vicinity of the joints is also well below 85% of yield when the pipe is subjected to design pressure, i.e., operating pressure plus 25 psi. In addition, the maximum stress remains below 85% of yield and von Mises stress below 95% of yield when the pipe is subjected to surge pressure.

2. For pipes near the bends subjected to the combined effects of internal pressure, temperature change, and soil strain from seismic wave propagation, the FEM results show that the pipe wall stresses are relieved as the elbow moves into the soil, and that the maximum stresses in the pipe wall near the elbows are well below the yield stress of the pipe wall. We also calculated the stresses in the joints and determined that yielding can occur for sharp bends with strains exceeding 0.2%; however, introduction of a bend radius of 2.5 D eliminated unacceptable yielding in excess of the 0.2% strain limit.

3. For the pipeline within the casing in the tunnel and the adjacent riser and tee, all subjected to the combined effects of internal pressure, temperature change, and soil strain from seismic wave propagation, the FEM results show that the design is acceptable. Pipe wall strains only exceed the 0.2% strain limit for a localized area on the extreme fiber of the inside surface of the pipe at the juncture with the crotch plates, and the crotch plate stresses remain well within the stress limit specified in the design criteria.

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Analysis and Behavior of Steel Pipe Welded Lap Joints in Geohazard Areas

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Abstract

Using advanced finite element simulation tools, which account for both geometric and material nonlinearities, the bending capacity of welded lap pipeline joints is investigated. Following the analysis of plain pipes, numerical results are reported for two steel typical grade 40 pipes, single-welded or double-welded, with D/t values equal to 150 and 240 respectively. Internal pressure effects are also examined, as well as the effects of bell-spigot gap. The results of the present study focus on the value of local strains developed at critical locations and are aimed at providing better understanding of welded lap joint behavior under extreme bending loading conditions. This may allow for the development of design methodologies in geohazard areas where welded joints are required, in order to safeguard the structural integrity of steel water pipelines imposed to severe ground-induced actions.

INTRODUCTION

Welded lap joints are often used in large-diameter steel pipelines for water transmission (AWWA M11). They are used as an alternative to straight butt-welded joints in water conduits, because of their lower construction cost and proven history of use. Those joints are constructed through a mandrel at one end of a pipe segment and expanding it to create an expanded cross-section of the pipe, often referred to as the "bell", into which the other end of the adjacent pipe segment, often referred to as "spigot", is inserted. Figure 1 shows the configuration of a welded lap joint; the bell and spigot ends are connected with a single or double full circumferential fillet weld. For pipelines larger than 36 inches (914 mm) in diameter, internal welds are frequently used, which permit person entry. In some cases, both internal and external welds are also considered.

The present paper is motivated by the need for determining the deformation capacity of welded steel pipelines for water transmission, constructed in geohazard areas. In such areas, e.g. areas with significant seismic activity, the pipeline can be subjected to severe permanent ground deformations, resulting from fault rupture, liquefaction-induced lateral spreading and subsidence, or landslide action. Under those extreme loading conditions, the pipe deforms well beyond the stress limits associated with normal operating conditions, whereas the structural performance of welded joints constitutes a key issue for pipeline structural integrity. In such geohazard areas, the main action on the pipeline is bending loading.

Most of the work on the structural capacity of welded lap joints has been directed in the investigation of their axial loading capacity, recognizing that for – in such a case – one should take into account their post-yielding performance. Failures at the vicinity of such joints have been observed either on the construction stage (Moncarz *et al*., 1987; Eberhardt, 1990), or after strong earthquake action (Meyersohn and O'Rourke, 1991; O'Rourke *et al*., 1995; Lund, 1996; Tutuncu, 2001). Because of the bell geometry, the stress path under axial compressive load has an eccentricity (Figure 1) and, therefore, an increase of longitudinal stress occurs, which may result in pipeline failure. Compressive tests on medium-scale specimens have been reported by the research group of Prof. T. D. O'Rourke at Cornell University (Jones, *et al*., 2004, Tutuncu and O'Rourke, 2006, Mason, 2006, Mason, *et al*., 2010). A finite element analysis of welded lap joints under axial compression has been presented by Tsetseni & Karamanos (2007), considering axisymmetric conditions. Full-scale tests on the compressive capacity of full-scale welded lap joints have been reported by Smith (2006), in an attempt to relate the experimental strength values with joint efficiency values, as reported in ASME B&PVC VIII. The tests specimens referred to 77.625-inch-diameter pipes with 0.323-inch thickness. It was concluded that the joint efficiency specified by the ASME code is quite conservative. The tensile response of welded lap joints has been examined analytically, based on longitudinal strip models (Eidinger, 1999; Brockenbrough, 1990; Moncarz, *et al*., 1987), or experimentally, with direct tension tests on small-diameter (medium-scale) pipe specimens (Mason *et al*. 2011). Finally, notable contributions on the practical use of welded lap joints, have been reported by Watkins *et al*. (2006), van Greussen (2008), and Bambei and Dechant (2009), Nevertheless, the mechanical behavior of welded lap joints under bending loading conditions, which is the major loading feature in geo-hazard areas, constitutes an open issue.

The study herein reports a finite element simulation of the structural performance of lap welded pipeline joints subjected to extreme bending loading conditions, resulting from those ground-induced actions, in the presence of internal pressure. Special attention is given on the geometry of lap joints, where the bell, the spigot and the weld cause non-uniform distribution of stress and deformation. Therefore, this welded connection can be regarded as an "initial imperfection" of the pipeline geometry from the "perfect cylinder" that may result in localization of deformation at the connection area, and reduce the structural capacity of the pipeline under bending.

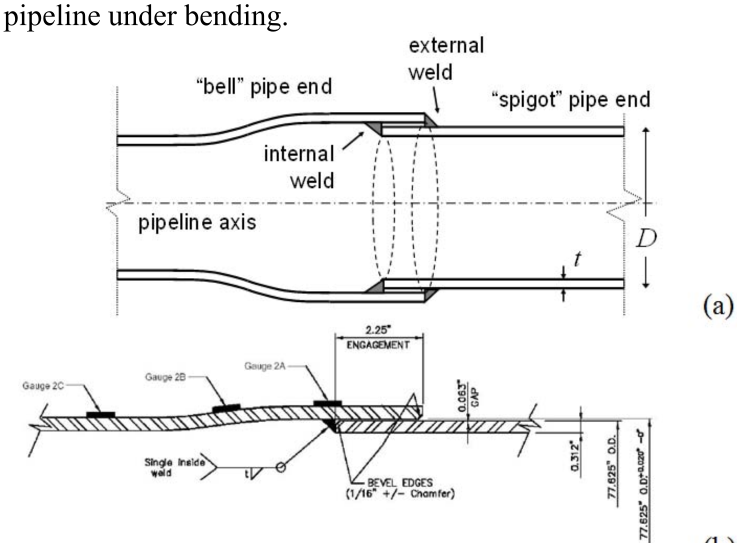


Figure 1. (a) Configuration of a welded lap joint; (b) Detail of an internal welded lap joint in a 77-inch-diameter pipe [Smith, 2006]

In the present simulation, the pipe and the welded joint are modelled with nonlinear finite elements, capable of describing pipe joint deformation in a rigorous manner. The use of those advanced numerical tools is aimed at developing a “numerical testing laboratory”, which can predict the structural behavior of welded lap joints in a reliable, yet cost effective manner. Numerical results are presented for plain steel pipes, and mainly for lap welded joints of steel pipes with diameter-to-thickness ratio values between 150 and 240, subjected to longitudinal bending and internal pressure. A comparison is also attempted with the mechanical performance of butt-welded joints, which are considered to restore the full continuity of the steel pipe, when are properly made per the AWWA C206 Field Welding Standard. Lap joints welds are considered as single-welded, mainly on the inside of the joint, or double-welded on both sides of the joint, and are subjected to bending loading, where the lap joint exhibits bulging and folding on the compressive side and stretching on the tension side. In the above cases, the effects of internal pressure of the welded pipe are considered, as well as the effects of the bell-spigot gap. Finally, the evolution of local strain at critical locations is monitored, in an attempt to evaluate the structural performance of those joints against fracture.

NUMERICAL MODELS

A three-dimensional numerical model is necessary to account for the nature of the present physical problem. The model is developed in finite element program ABAQUS/Standard. Both the bell and the spigot are modelled with three-dimensional nonlinear four-node reduced-integration shell elements, which have been quite efficient in simulating buckling and post-buckling response of thin-walled cylindrical members (Vasilikis *et al*. 2014). Those elements can account for geometric nonlinearities, such as large deformations and buckling, as well as the nonlinear (inelastic) behavior of steel pipe material well beyond the elastic regime. To account accurately for possible contact between the bell and the spigot parts, the inner surface of the bell and the outer surface of the spigot are considered as reference surfaces of the shell model under consideration. In those surfaces, appropriate contact conditions have been imposed, which present penetration of one surface to the other, but allows for their separation. Furthermore, to account rigorously for the geometry of the weld, a full-circumference ring is modelled using solid elements with a 45-degree triangle, as shown in Figure 2. The triangular ring is connected to the bell and the spigot with appropriate kinematic conditions. The numerical model does not account for any symmetry, despite the fact that – initially – the deformation is symmetric with respect to the plane of bending. However, both the welded lap joint geometry and the buckling shape of the pipe are not symmetric, and therefore, a full three-dimensional model should be considered.

Two types of models are developed, one for “plain pipes”, i.e. pipes that do not contain a welded connection, and one for pipes with welded lap connections. The former models can also be regarded as representative for butt-welded pipe joints as well, assuming that butt welds restore fully pipeline continuity between two adjacent pipe segments. Of course, special issues on butt-welded joints, such as “high-low mismatch” or different material properties on each side of the joints, are not examined.

For each case analysed, the numerical model is 10-diameter-long, and the finite element mesh is considered quite dense in the area where buckling is expected. For the “plain pipe” models, a 2-diameter-long central section is modelled with a dense mesh, where the element size is equal to 1/200 of the pipe diameter, whereas the size of the elements in the circumferential direction is equal to 1/56 of the pipe diameter. It is noted that, from shell buckling theory, a good estimate for the half-wave length in the axial direction of the pipe can be obtained from $1.22\sqrt{Dt}$. This means that for a pipe with D/t equal to 240, 16 elements are contained within each half-wavelength, a number which is considered very satisfactory for the purposes of the present analysis. A coarser mesh is considered for the pipe parts away from this central area. A typical mesh for "plain pipe" models is shown in Figure 3. At the two ends of the pipe model, two "fictitious" nodes are introduced on the pipe axis, connected to the nodes of the end-section with appropriate kinematic conditions, referred to as "kinematic coupling" in ABAQUS. The pipe model is considered simply-supported in those two ends, and bending is applied with two opposite bending moments at the end nodes. In the case of "welded lap joint" models, the welded connection is located in the middle of the pipe segment, and a section of pipe length equal to about 1.4 pipe diameters containing the welded lap joint is modelled with a dense finite element mesh, similar to the one used for the central section of the "plain pipe" models.

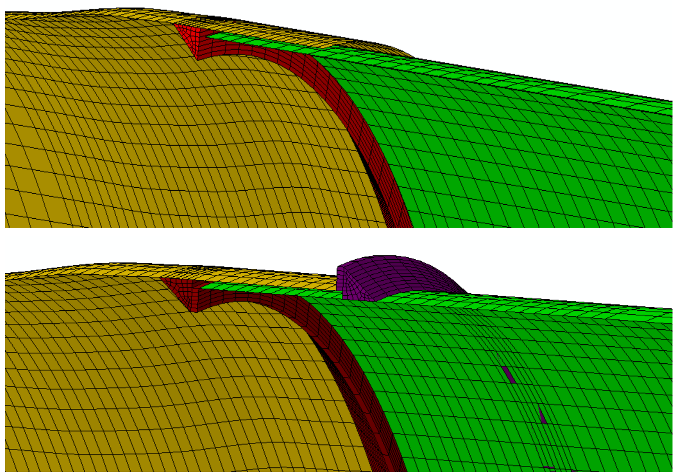


Figure 2. Finite element models for welded lap joint simulation; geometry of the weld area; internal weld (top) and double weld (bottom).

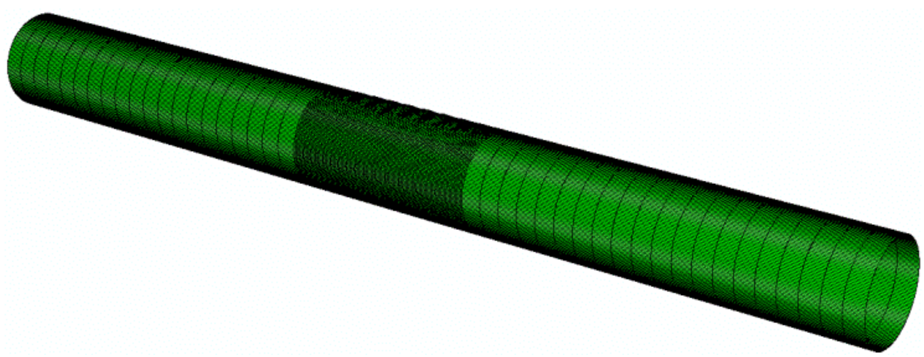
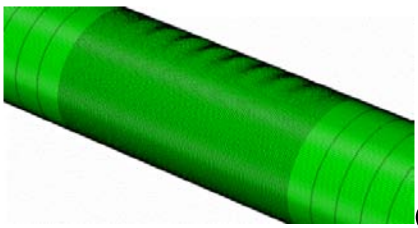
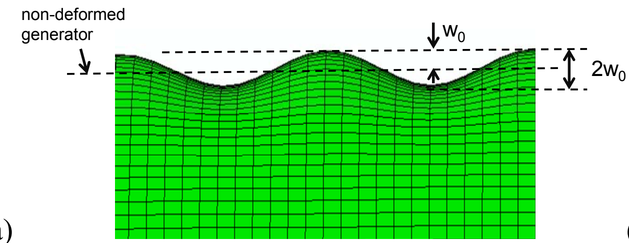


Figure 3. Typical finite element mesh of a "plain pipe" model; mesh is finer in the central section of the pipe, where buckling is expected to occur.

In the steel pipes under consideration, with diameter-to-thickness values higher than 150, the primary mode of failure under bending is bucking of pipe wall at the compression side. In plain pipes, to avoid numerical convergence problems in the nonlinear finite element analysis, an imperfection should be assumed so that transition to the buckled shape is triggered. This imperfection is a wrinkling pattern, in the form of the first buckling eigen-mode of the pipe subjected to bending, obtained through a bifurcation (linear buckling) analysis in ABAQUS, before the nonlinear analysis is performed, as shown in Figure 4a. The buckling eigen-mode (wrinkling pattern) is expressed in terms of nodal displacements, and before added to the cylinder geometry, it is multiplied by an appropriate constant, so that wrinkling amplitude w_0 is controlled (Figure 4b). In the case of welded lap joints, consideration of such an imperfection is not necessary; the presence of the welded lap joint constitutes an "imperfection" for the pipe under bending.


(a)



(b)

Figure 4. (a) Wrinkling pattern used as initial imperfection in the analysis of plain pipes; (b) wrinkling amplitude w_0.

NUMERICAL RESULTS

Two different steel pipes with different diameter-to-thickness ratio D/t are examined. The first pipe, denoted as "Pipe I", has a 56.25-inch diameter and 0.375-inch wall thickness (D/t equal to 150), whereas the second pipe, denoted as "Pipe II", has a 77.625-inch diameter and 0.323-inch wall thickness (D/t equal to 240). The material of both pipes is ASTM 1018 grade 40 steel (Smith, 2006), and the stress-strain curve is shown in *Figure 5*. The yield stress is 303 MPa (43,900 psi), as reported by Smith (2006), there is a plastic plateau up to 1.5% of strain, typical for structural steels, and subsequently, strain hardening occurs, with a plastic modulus equal to approximately 1/500 of Young's modulus.

Results for the effects of internal pressure and the influence of the gap size between the bell and the spigot are presented for lap joints. The results for welded lap joints refer to both "internal-welded" and "double-welded" pipe joints. For each case, the moment-deformation relationship is determined. Local buckling (bulging) and the subsequent folding of pipe wall due to excessive compression at the "intrados" of the bent pipe is simulated explicitly with the finite element solution. In addition, local strains are measured in critical locations for different levels of loading, so that the possibility of joint failure is detected.

Results for plain pipes

The response of plain pipes under bending loading, in the absence of internal pressure, is shown in Figure *6*a and Figure *6*b, for Pipe I and Pipe II respectively, in terms of moment-curvature diagrams, for different values of initial wrinkling imperfection amplitude. The reported value of curvature is computed as the ratio of the relative rotation of the two end sections of the pipe model over the model length and this can be regarded as a "global measure" of normalized rotation of the bent pipe segment under consideration. The values of bending moment are normalized by the fully-plastic moment $M_P = \sigma_Y D^2 t$, whereas the values of curvature are normalized by the "curvature-like" parameter $k_I = t/D^2$, following the relevant suggestion in Karamanos & Tassoulas (1996). The results show a significant reduction of structural strength for increasing amplitude of initial imperfection. This reduction is shown in graphical form in Figure 7a and Figure 7b. The buckling shape of those pipes is shown in Figure 8 and Figure 9, indicating the formation of a major buckle, located symmetrically with respect to the plane of bending, and several secondary or "side" buckles. This refers to a "diamond-shape" buckling pattern, typical for thin-walled shells subjected to compressive loading (Vasilikis *et al*., 2014).

The presence of internal pressure influences bending response. The corresponding moment-curvature diagrams for Pipes I and II are shown in Figure 10a and Figure 10b, indicating an increase of bending capacity in the presence of pressure. The buckling shape in the presence of pressure, shown in Figure 11, is characterized by "bulging", which is typical for internally pressurized cylinders.

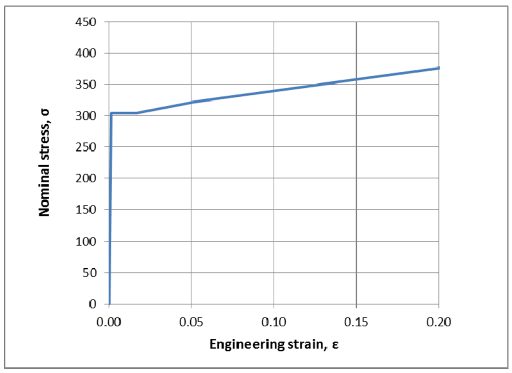


Figure 5. Stress- strain curve of ASTM 1018 grade 40 steel used in the present analysis; yield stress is equal to 303 MPa [43,900 psi].

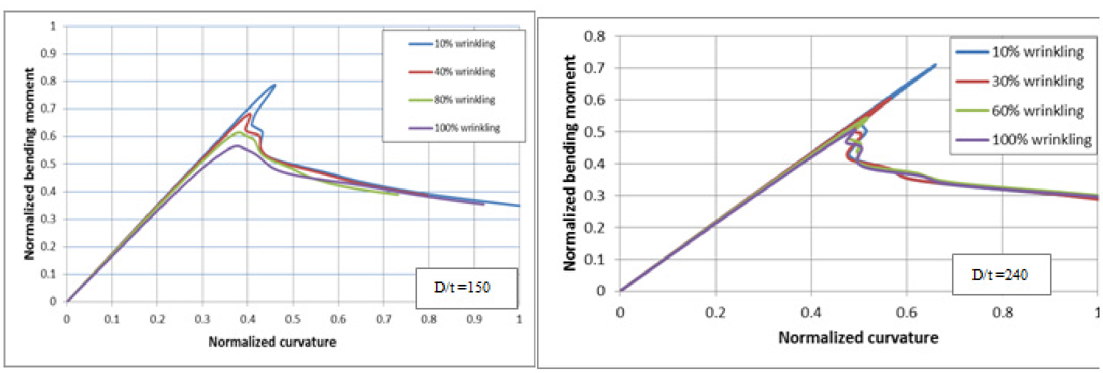


Figure 6. Moment – curvature diagrams for plain pipes for different values of initial imperfections; (a) $D/t =$ 150 ; (b) $D/t =$ 240.

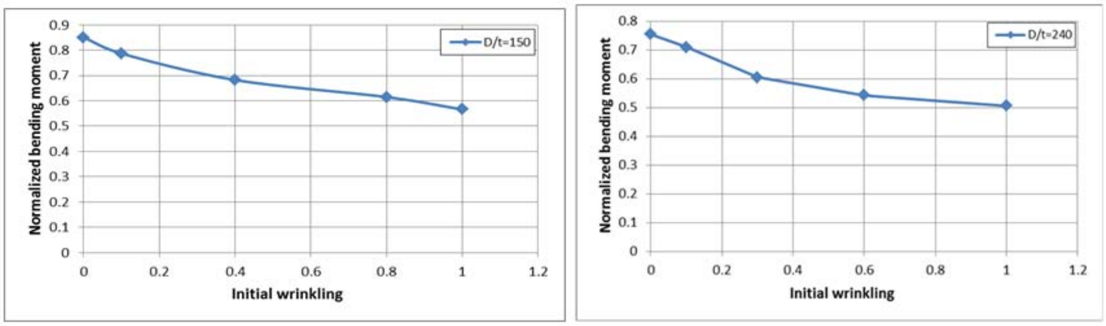


Figure 7. Reduction of bending strength with increasing amplitude of initial imperfection for plain pipes; (a) Pipe I: $D/t =$ 150 ; (b) Pipe II: $D/t =$ 240.

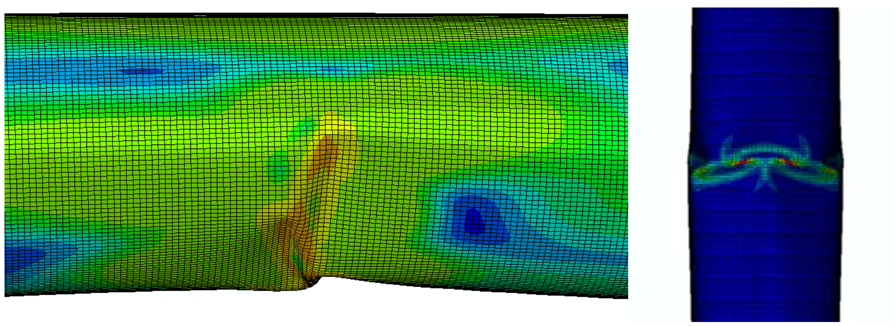


Figure 8. Plain Pipe I with D/t equal to 150 and small initial wrinkling (amplitude equal to 10% of pipe thickness).

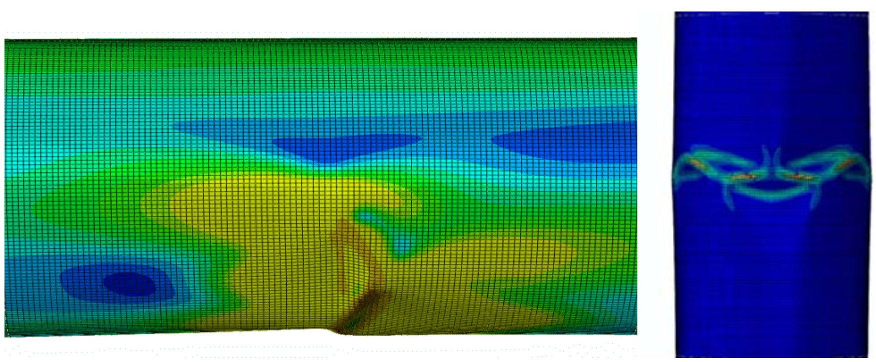


Figure 9. Plain Pipe II with D/t equal to 240 and small initial wrinkling (amplitude equal to 10% of pipe thickness).

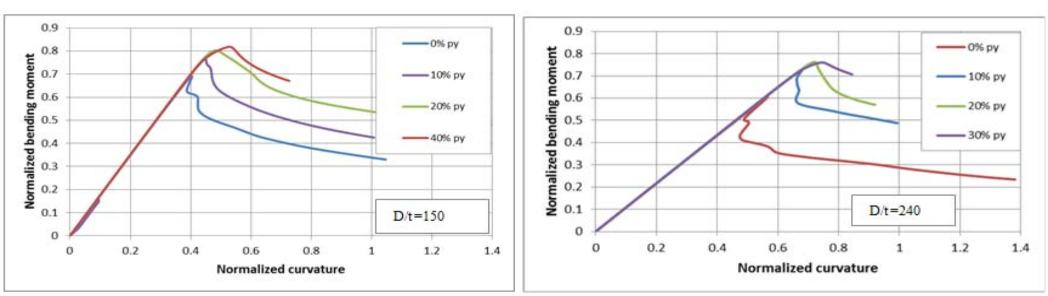


Figure 10. Bending response in the presence of internal pressure of Pipes I and II; initial wrinkling amplitude equal to 30% of pipe thickness.

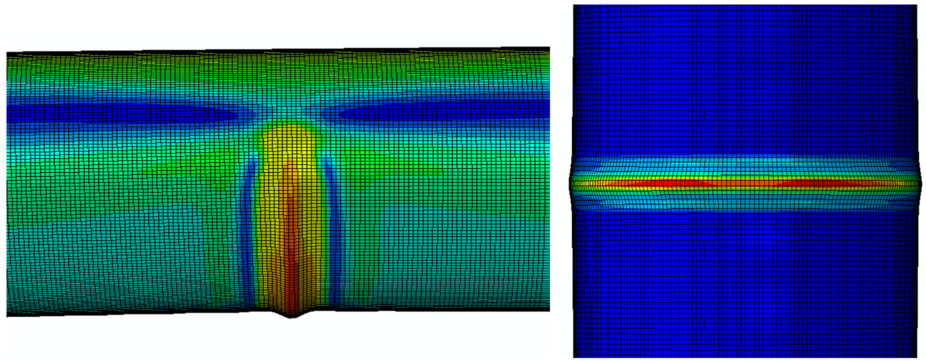


Figure 11. Buckling shape of a plain pipe in the presence of pressure ($D/t =$ 240).

Results for welded lap pipe joints

The bending response of pipe segments containing welded lap joints is different than the response of “plain pipes”. The presence of a welded lap joint, because of the bell geometry, introduces an initial geometric “imperfection” in the pipe. Under bending loading, at the compression side of the pipe, the welded lap joint is significantly deformed, in the form of localized wrinkling and folding, reducing the bending capacity of the welded pipe with respect to the capacity of a plain pipe. Figure *12*a and Figure *12*b show the response of internally-welded and double-welded joints for Pipe I and Pipe II respectively, in terms of the corresponding moment-curvature diagrams. The results show that, in both cases, the ultimate strength of the welded lap joints is lower than the capacity of the corresponding plain pipe. Furthermore, the use of double welds, instead of single (internal) welds, has a minor effect on the structural behavior. Figure 13 shows the deformed shape of an “internal welded” pipe, subjected to bending; significant localized deformation occurs at the weld area, associated with “wrinkling” and “folding”, and this local deformation responsible for pipe failure and reduction of structural strength. The shape of the deformed welded joint is quite similar to the ones shown in Figure 8 and Figure 9.

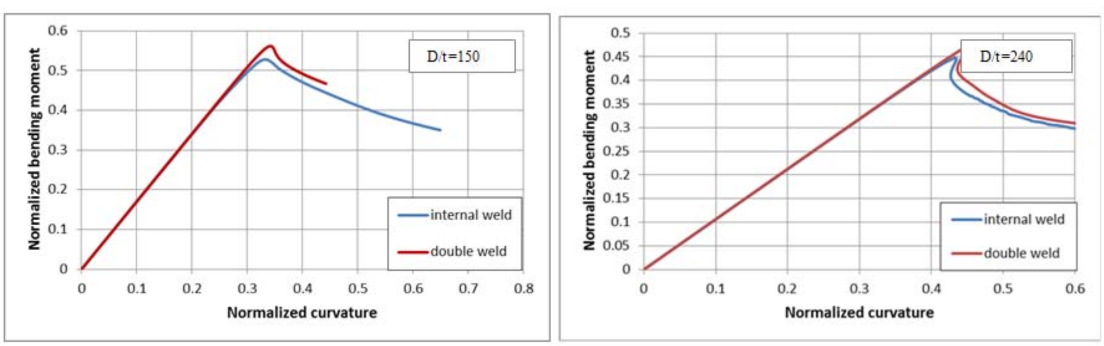


Figure 12. Moment-curvature diagrams for internally-welded and doubly-welded joints; (a) Pipe I, $D/t =$ 150 ; (b) Pipe II, $D/t =$ 240.

Figure 14 and Figure **15** show the effects of internal pressure on the bending response of welded lap joints. The effect is similar to the one observed for plain pipes: the capacity increases when the level of internal pressure is raised. Furthermore, the wrinkling shape for higher levels of pressure is characterized by the "bulging" pattern, as shown in Figure 16, also observed in Figure 11 for plain pipes. Finally, the results in Figure *17* show that the effects of gap size for the range of gap values considered are rather minimal. The maximum gap size is equal to 0.125 in, which is the maximum specified by AWWA C206.

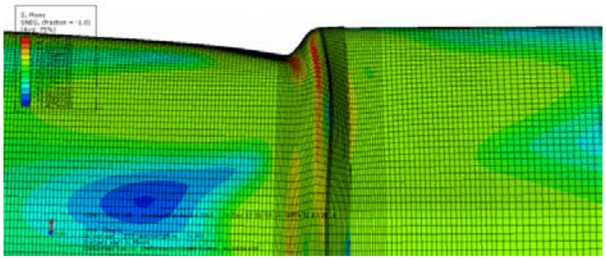


Figure 13. Deformed shape of an "internal weld" lap joint subjected to bending loading, characterized by localization of deformation; Pipe I, $D/t =$ 150, zero pressure.

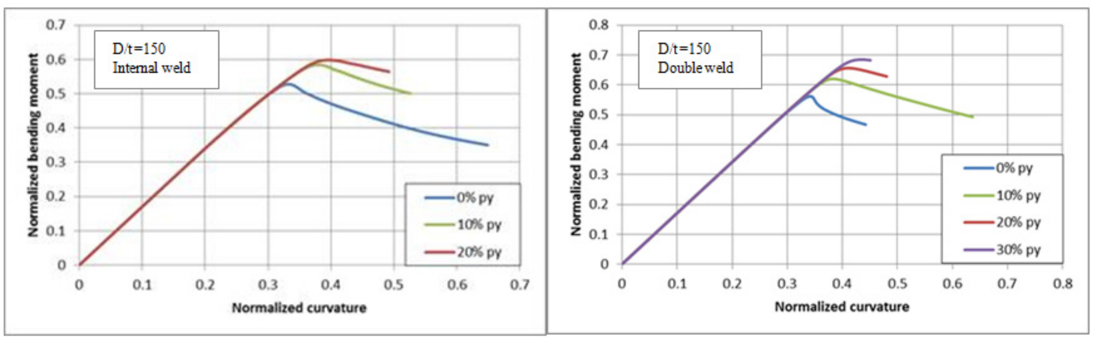


Figure 14. Effect of internal pressure on the bending response; (a) internally-welded joints and (b) doubly-welded joints (Pipe I, $D/t =$ 150).

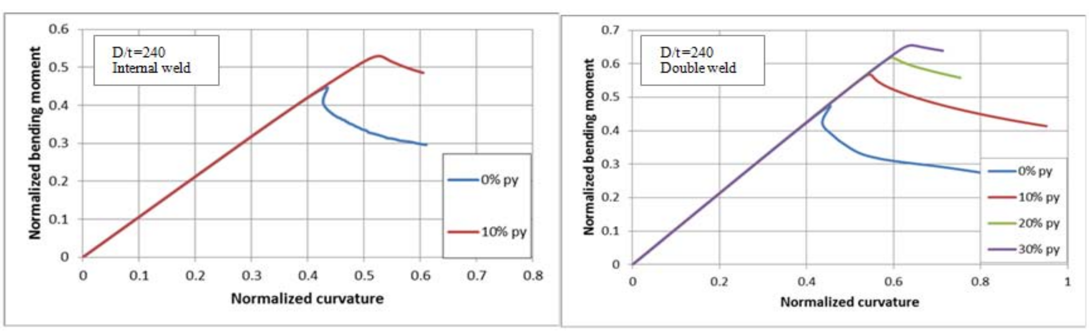


Figure 15. **Effect of internal pressure on the bending response; (a) internally-welded joints and (b) doubly-welded joints (Pipe II, $D/t =$ 240).**

Assessment of welded lap pipe joints

The above results focused on the global mechanical behavior of welded lap joints subjected to severe bending loading, in terms of the moment-rotation response of a finite length pipe segment, containing the welded lap joint. The "buckled shapes" depicted in

Figure 13 or in Figure 16, constitute a limit state for the welded pipe. However, such a wrinkled shape may not be necessarily associated with total failure and loss of containment. To assess the structural integrity of welded pipe joints against pipe wall fracture, it is necessary to proceed beyond the above analysis, monitoring the evolution of local strain at critical locations. The cases considered refer to Pipe I ($D/t =$ 150) with "internal weld" and "double weld". Two cases are considered for "internal weld": one case is without pressure, referred to as Case A, and one with pressure 20% of yield pressure, referred to as Case B. In Case A, the critical locations, at which maximum tensile strain occurs, can be identified as follows, as shown in Figure 18a:

(1) Compression side: pipe wall folding at the ridge of buckle (top surface).
(2) Compression side: pipe wall folding at the ridge of buckle (bottom surface).
(3) Compression side: weld connection at bell end.
(4) Compression side: weld connection at spigot end.
(5) Tension side: weld connection at spigot end.

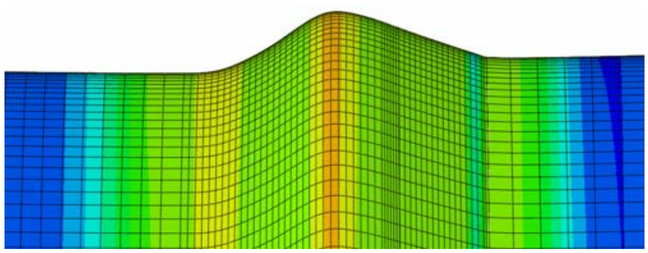


Figure 16. Effect of pressure on deformed shape of welded lap joint subjected to bending loading; compression side of the pipe ($D/t =$ 240; internal weld).

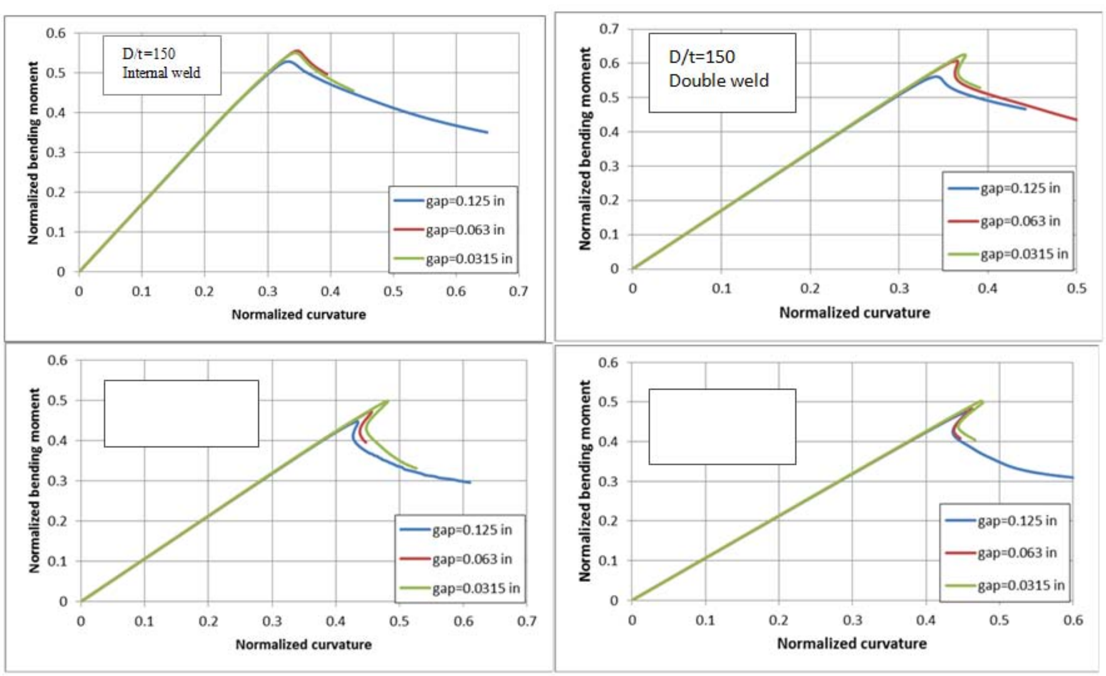


Figure 17. Effect of gap between the bell and the spigot on the structural performance of welded lap joints Pipe I at top; $D/t =$ 150; and Pipe II at bottom; $D/t =$ 240).

The evolution of tensile strain at the above locations is shown in Figure 19. The results show that up to buckling, the level of local strain is rather low. On the other hand, upon the occurrence of buckling, the value of local strains increases very rapidly. In locations (1) and (2), this is attributed to local folding of the pipe, whereas in locations (3), (4) and (5), the discontinuity due to the presence of the weld is responsible for this strain raise. Based on the numerical results, the maximum longitudinal strain is located at the ridge of the buckling, at location (1). Soon after the maximum strength of the joint is reached, at a value of normalized curvature equal to 0.33, the longitudinal strain increases rapidly and reaches values close to 10%, which implies that, at this location, pipe wall may fracture. Furthermore, the maximum hoop strain occurs at location (3), because of significant displacement radial ("bulging").

Furthermore, the critical locations of the pressurized welded lap joint of Case B are shown in Figure 18b:

(1) Compression side: pipe wall folding at the ridge of buckle (top surface).
(2) Compression side: pipe wall folding at the ridge of buckle (bottom surface).
(3) Compression side: weld connection at spigot end.
(4) Tension side: weld connection at spigot end.

The results for the local strains are depicted in Figure 20. From the qualitative point-of-view, the numerical results are quite similar to the ones presented in Figure 19 for the unpressurized Case B. In this case, the tension side appears to be most critical.

Two cases are also considered for "double weld": Case C is without pressure, and Case D corresponds to pressure 20% of yield pressure. The corresponding critical locations, at which maximum tensile strain occurs, are shown in Figure 21 and are similar to the ones for Cases A and B and the evolution of local strains at those locations are shown in Figure 22 and in Figure 23.

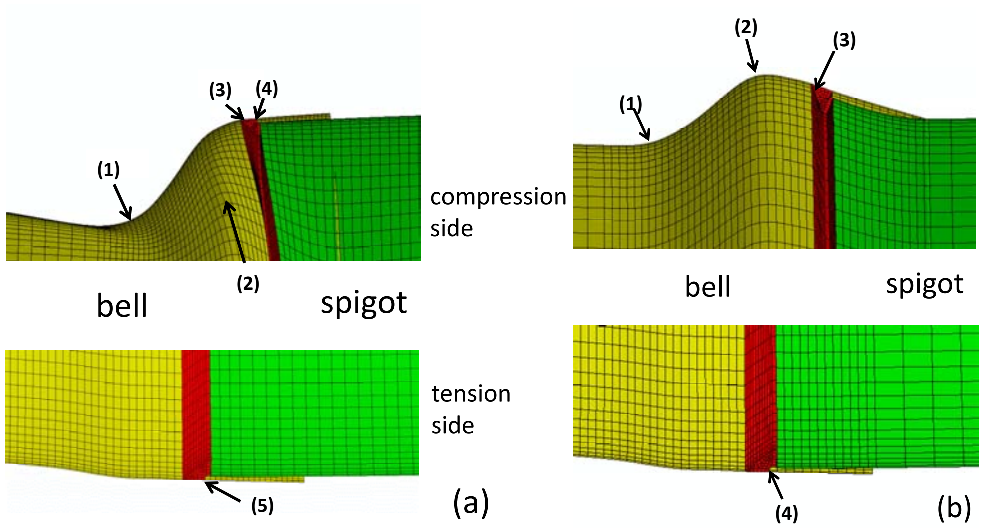


Figure 18. Critical locations in welded lap joints subjected to bending Pipe I (D/t = 150; "internal weld"); (a) zero pressure and (b) pressure equal to 20% of yield pressure.

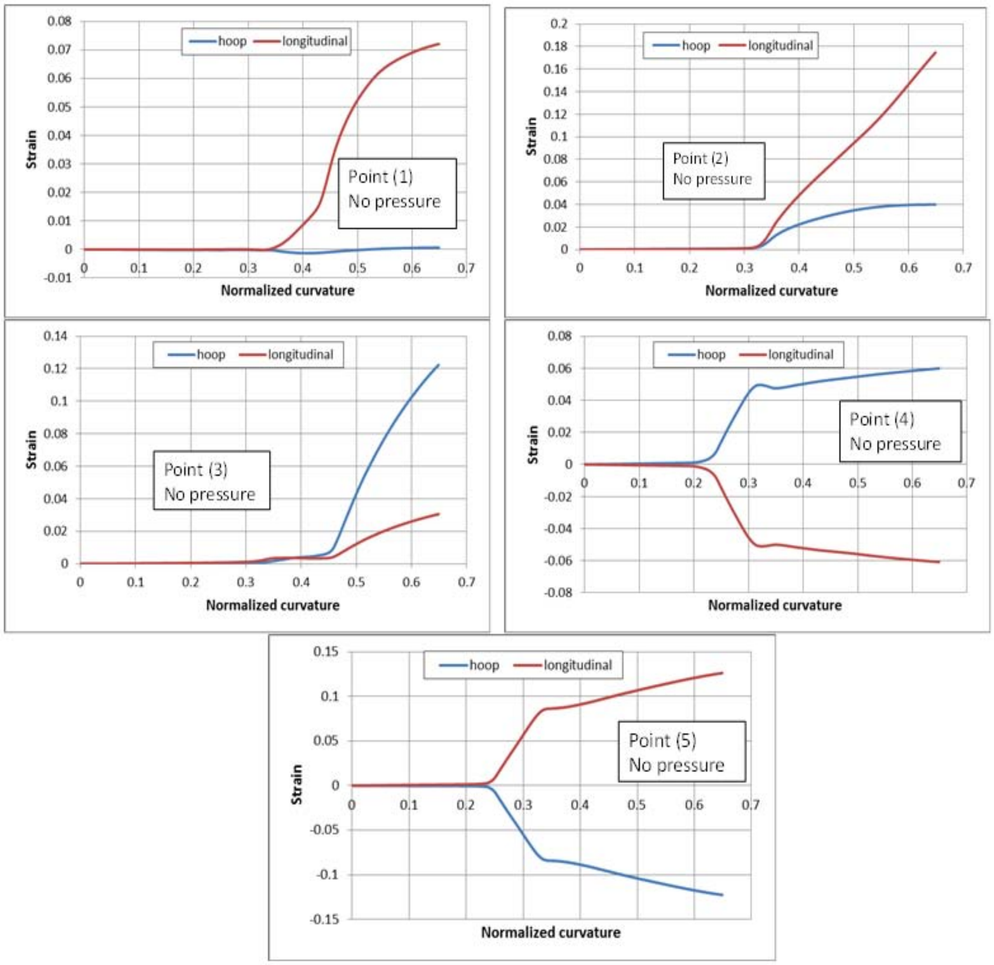


Figure 19. **Evolution of local strain at the five critical locations of welded lap joints subjected to severe bending (Pipe I; $D/t =$ 150; "internal weld"; zero pressure).**

DISCUSSION AND CONCLUSIONS

The present work reported the development of rigorous numerical (finite element) tools, which enable simulation of the structural performance of welded lap joints under severe bending loading. Both "internal weld" and "double weld" joints have been examined. It is found that the presence of internal pressure affects joint behavior and influences significantly the value of local strains at critical locations. The most critical locations are in the vicinity of the weld area (both in the compression and tension side), as well as the ridge of the buckle (in the compression side).

The finite element results indicated that welded lap joints subjected to bending, are capable of sustaining significant deformation (rotation) after the occurrence of buckling, without fracture and loss of containment. Therefore, they can be used in areas where severe ground-induced actions are expected, e.g. in fault crossings, in liquefaction areas and in areas of potential landslide. In those areas, welded lap joints can be an efficient solution, in comparison to butt-welded full-penetration joints. In geohazard areas where low level ground movement is expected, rubber gasketed joints can be utilized. More specifically, used in combination with welded lap joints, the gasket joint can be used advantageously in the design of a pipeline in certain geohazard areas (Karamanos *et al*. 2014).

Finally, it should be emphasized that the bending action considered constitutes an extreme external action for the pipeline. Therefore, classical "stress-based design", which considers stress allowables, as a percent of yield stress, is no longer applicable. In such a case, a nonlinear analysis capable at simulating large local deformations of the welded lap joint is necessary.

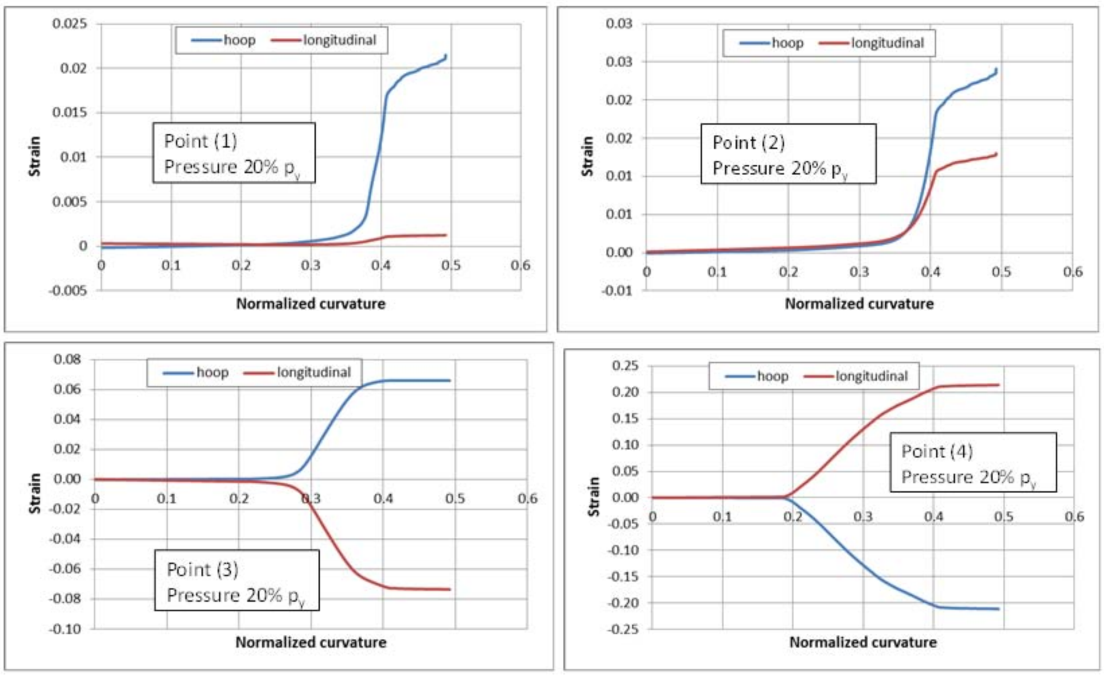


Figure 20. Evolution of local strain at the four critical locations of welded lap joints subjected to severe bending (Pipe I; $D/t =$ 150; "internal weld"; internal pressure equal to 20% of yield pressure).

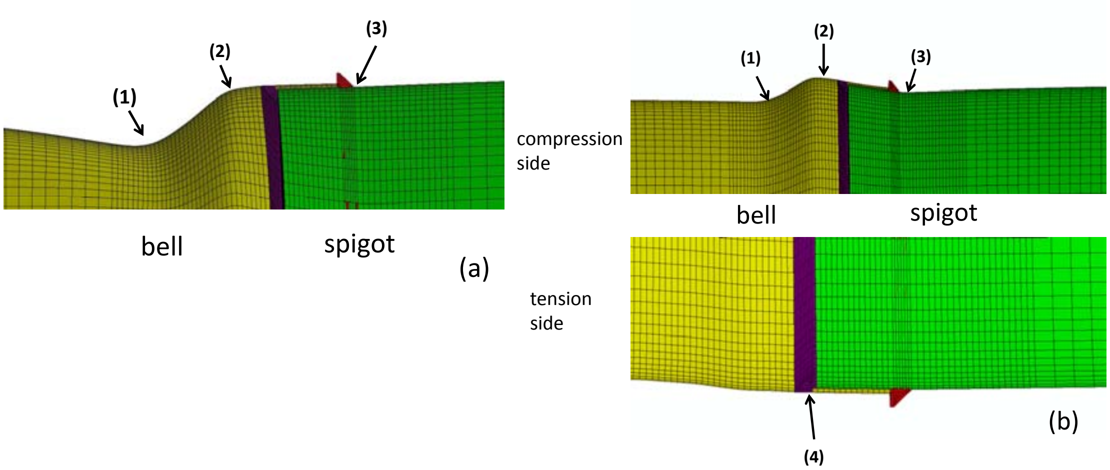


Figure 21. Critical locations in welded lap joints subjected to bending Pipe I ($D/t =$ 150; "double weld").

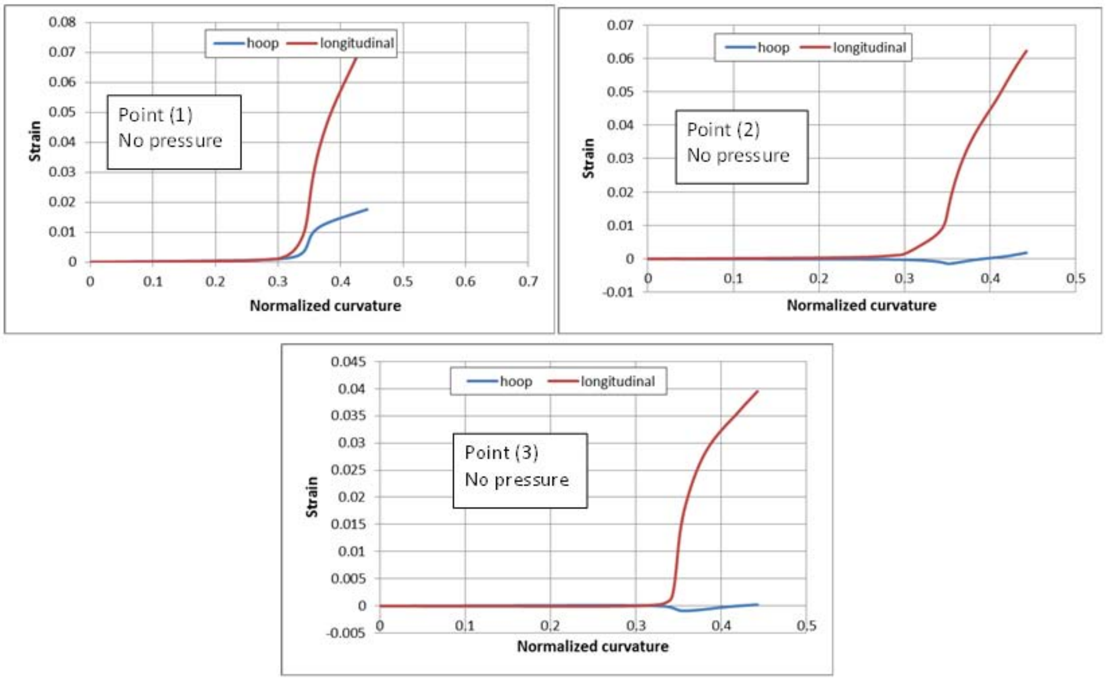


Figure 22. Evolution of local strain at the three critical locations of welded lap joints subjected to severe bending (Pipe I; $D/t =$ 150; "double weld"; zero pressure).

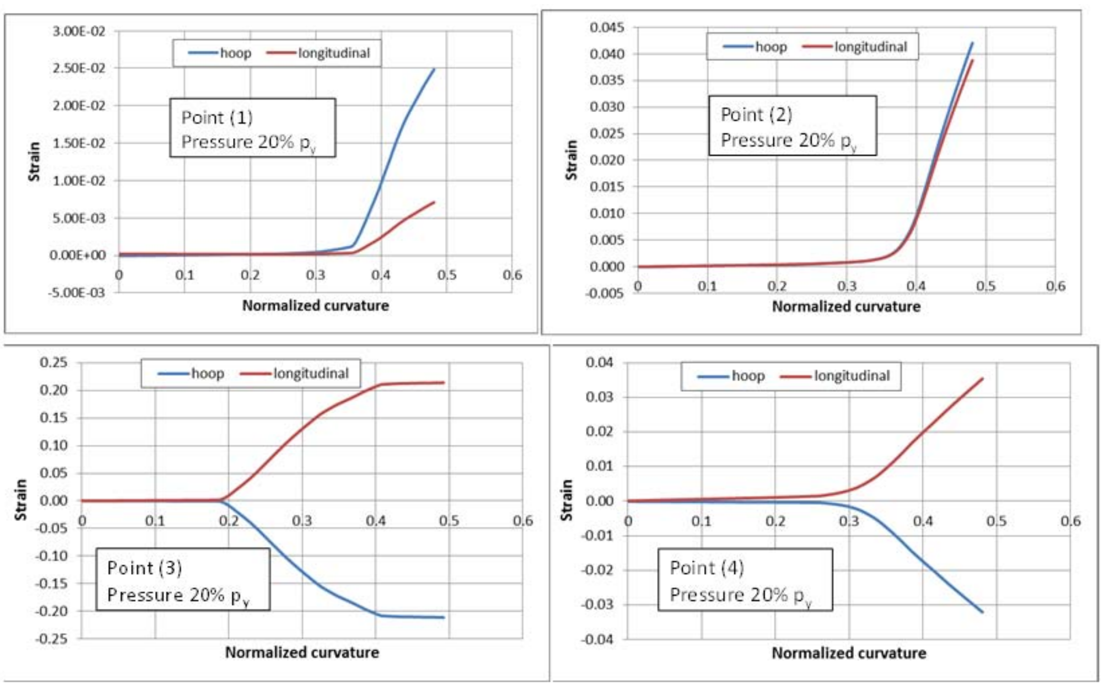


Figure 23. Evolution of local strain at the four critical locations of welded lap joints subjected to severe bending (Pipe I; $D/t =$ 150; "double weld"; internal pressure equal to 20% of yield pressure).

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Performance of Polypropylene Corrugated Pipe in North America

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Abstract

Polypropylene (PP) corrugated pipe is one of the newest products for sanitary and storm sewer applications in North America. Polypropylene pipe has been used for decades in Europe, but was not used in United States and Canada until the past 5 years, when new ASTM International, AASHTO and CSA standards were developed and approved for the applications more typical to North America. Polypropylene offers unique benefits over previous plastic materials used for sanitary and storm sewers. It has very high stress crack resistance, essentially negating any concerns with the type of stress cracking associated Stage II plastic pipe failure. It also has very good impact resistance and a high modulus of elasticity giving it both better performance at low temperature installations and a higher pipe stiffness. This increase in modulus also provides corrugated PP pipe with better beam strength, which helps to mitigate field deflection. In a very short period of time, corrugated polypropylene pipe has been specified and installed on numerous major sanitary and municipal drainage projects. This paper will cover the specific aspects that make polypropylene a unique material for pipeline construction. The various national standards shall be discussed with the emphasis on the performance criteria that these products must be tested to and meet as part of their certification. Some large infrastructure projects will also be highlighted to demonstrate its current acceptance and use.

INTRODUCTION

Polypropylene is a thermoplastic polymer that is used in a wide variety of applications, predominately those for non-structural commercial products such as textiles (carpets), toys, automotive parts (bumpers, molded components), containers (food, shampoo), etc. Polypropylene is similar to polyethylene in that it has the same carbon to carbon backbone chain with the only exception being the addition of a methyl group, CH_3 molecule, in lieu of an hydrogen, H, molecule on alternate carbon molecules (Figure 1). It is known as being a relatively rugged material that has strong resistance to impact. It, however, is this property that historically made it a poor choice for long-term structural products, such as pipe. For most of its common applications, polypropylene is blended with rubber (EPDM) to provide impact resilience and is characterized as an impact copolymer. The best long-term material mechanical properties such as modulus of elasticity are obtained with a homopolymer, but the associated high crystallization of this polymer results in it having very poor impact resistance. The task for pipe designers and manufacturers is, therefore, to develop a polypropylene that maintains a relatively high long-term modulus while maintaining as high an impact resistance as possible.

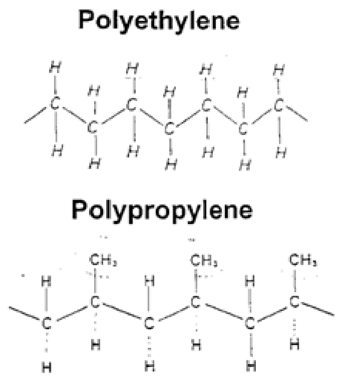

Figure 1. Polyethylene and Polypropylene Molecular Chains

Polypropylene compounds used for pipe production have many of the strengths of high density polyethylene (HDPE) compounds such as high corrosion and abrasion resistance. Unlike PVC compounds, it can be heat fused in lieu of gluing and has no chemical leaching issues, and PP has vastly higher impact resistance than PVC. It also is not subject to any stress-cracking, which is one of the critical long-term concerns with HDPE materials. Although the modulus of elasticity and tensile strength of pipe grade polypropylene are lower than PVC, they are considerably higher than HDPE. The main issue which must be addressed with PP pipe is oxidation.

Polypropylene is subject to oxidation degradation from exposure to heat and ultraviolet radiation such as sunlight. In pipe, this degradation manifests itself as a network of fine cracks that increase in depth and severity based on time of exposure. The most critical time for pipe is when it is stored outside prior to burial. In general, the antioxidant package used in the manufacturing of PP pipe will provide approximately 2-years of outside storage protection, but this time varies depending in part on the location of the storage. For example, storage outside in Arizona would be more severe than Ohio due to the sunlight intensity and days of full sun exposure. Oxidation appears as chalking or whitening of the pipe. Carbon black, commonly used in HDPE pipe, or titanium dioxide, commonly used in PVC pipe, is typically used for ultraviolet protection for PP pipe.

Polypropylene also retains a heat history. Reheating or reprocessing of PP depletes the antioxidant package and reduces its OIT (Oxidation Induction Temperature) resistance. If PP is to be reprocessed or recycled, it may need to have an additional antioxidant included in its processing if its minimum OIT requirements are not obtained.

Polypropylene has unique benefits that compare extremely favorably to other plastic materials, and for this reason, it has become the product of choice for pipe in many locations around the world. These engineering, environmental and performance benefits are reflected in the recent standards that have been developed by ASTM International, AASHTO (American Association of State Highway Officials) and CSA (Canadian Standards Association).

DESIGN

Prior to the development of national and international standards for PP pipe, its design strengths and limits had to be evaluated. Although PP resins had been used for many years, there were virtually no PP materials available ten years ago with long-term mechanical engineering properties, specifically structural performance values for a 50-year design life or longer. All thermoplastics are time dependent materials with properties that change when exposed to a constant stress (i.e. creep). These values must be determined in order to properly evaluate the long-term structural integrity of pipe.

The initial modulus of elasticity (E) and tensile strength (F_u) of PP are 175,000 psi and 3,500 psi, respectively. These compare very favorably to HDPE which have initial values for E and F_u of 110,000 psi and 3,000 psi. For this reason, PP pipe has much higher initial pipe stiffness when compared to HDPE pipe. Initial material mechanical properties, however, do not determine the integrity of the pipe's long-term structural performance.

PP pipe, like HDPE and PVC thermoplastic pipe, is designed in accordance with the AASHTO LRFD Bridge Design Specifications. These design specifications only address non-pressure pipe applications, the cell classifications for the AASHTO PP, HDPE and PVC pipe materials are 12364C (ASTM D1784), 435400C (ASTM D3350) and the compound performance requirements in AASHTO M330, respectively. One of the critical design requirements is to obtain appropriate 50-year minimum material mechanical properties for E and F_u. The 50-year modulus of elasticity and tensile strength values for PP pipe are 27,000 psi and 1,000 psi. The obtaining of these values proved to be very difficult with the PP materials that were provided by resin manufacturers in North America ten years ago. Since most PP resins went into short-term applications that necessitated high impact resistance (i.e. PP compounds with high rubber content), by design long-term structural performance was minimized. The PP pipe that has been used in Europe for over 20-years also did not stipulate these criteria since their applications were typically smaller diameter pipe under shallow installations, where the applied stresses are much less than those experienced under the deeper sewer applications in North America. The result was the lack of any PP resins that could be used for PP pipe applications in North America.

Through a multi-year research effort conducted with pipe manufacturers, various resin manufacturers, consulting engineering firms, DOT's and universities, a group of resins were developed to meet the required AASHTO 50-year material mechanical properties. Some of the interesting facets which came out of this work were the performance relationships between PVC, HDPE and PP pipe. PP pipe has some of the critical strengths of both HDPE and PVC with few of the weaknesses associated with each product.

PP and HDPE pipe both have very good impact resistance, which is superior to PVC pipe. The pipe stiffness of PVC and PP are significantly better than HDPE due to their higher initial modulus of elasticity. The leaching concerns with PVC that led to the greater use of PP pipe in Europe do not exist with PP and HDPE pipe. Neither PVC nor PP pipe have any stress cracking concerns in non-pressure (gravity flow) sewers typically associated with HDPE pipe (ASTM F2306). The PVC and PP pipe beam strength, which helps mitigate deflection, is greater than that of HDPE pipe. The high hoop stiffness of HDPE and PP pipe, due to their similar effects associated with creep, allows for more circumferential shortening in the pipe thus increasing the soil arching over the pipe and reducing the load applied to the pipe. In relative terms, HDPE and PP pipe can be buried deeper more effectively than PVC pipe. PVC and PP pipe do not have the same ultraviolet resistance as HDPE pipe, since HDPE pipe uses the best UV inhibitor, carbon black. The main concern with PP pipe, as mentioned earlier, is maintaining adequate OIT protection.

The national standards that cover PP pipe provide a composite compilation of all the factors associated with PP design and performance. Due to polypropylene's relatively new entrance into the pipe market, these PP standards are be the most comprehensive evaluation of any pipe material as they have benefitted from a wealth of knowledge developed over the years for other thermoplastic products.

STANDARDS

There a currently five standards that address polypropylene materials for sanitary and storm sewer pipe. These are ASTM F2736, ASTM F2764, ASTM F2881, AASHTO M330, and CSA 182.13. The structural design criteria for PP pipe are covered under the thermoplastic design requirements in Section 12, AASHTO LRFD Bridge Design Specifications. Unlike most material standards, all these noted PP material standards contain design requirements referencing the AASHTO design methodology in addition to material testing requirements.

ASTM F2736 covers 6-inch through 30-inch PP pipe manufactured with either a single wall corrugation (i.e. corrugated interior and exterior) and dual wall (i.e. smooth interior wall and corrugated exterior wall) (Figure 2). The applications under F2736 apply to storm/land drainage (single wall), as well as, storm sewers and sanitary sewers (dual wall). A minimum pipe stiffness of 46-psi is required for all pipe diameters. This standard has the same material requirements for all applications and specifically calls out a minimum OIT requirement of 25 minutes when tested in accordance with ASTM D3895. The 50-year long-term tensile and modulus of elasticity are specified and required to be verified in accordance with ASTM D2990, which is a 10,000 hour creep test. The 10,000 hour creep rupture tensile strength and creep modulus tests are the internationally recognized test protocol for evaluation of the long-term properties of thermoplastics. In addition to the material test requirements, the standard requires the evaluation of the structural design safety factors for the pipe profile, geometry, wall centroid, wall area, wall moment of inertia, and the material strain limits in accordance with the AASHTO LRFD Bridge Design Specifications, Section 12, Buried Structures.

ASTM F2764 covers triple wall 30-inch through 60-inch PP pipe for sanitary sewer applications. This pipe has a smooth interior wall and smooth exterior wall (Figure 2) and requires a 46-psi minimum pipe stiffness of all diameters. In addition to all the material and structural requirements specified for F2736, it also mandates watertight joints tested to ASTM D3212. This laboratory water and vacuum test at 10.8 psi pressure test under straight and deflected positions is the standard test to certify acceptable sanitary sewer pipe joints. ASTM F2764, however, requires a much more stringent long-term evaluation of the joint's integrity. Since all thermoplastic pipe have time dependent mechanical properties and creep under a constant stress, a short-term D3212 joint test may not indicate the pipe's true long-term performance. F2764 requires an additional 1000 hour proof-of-design joint test with the pipe held under a constant stress. At 1000 hours, approximately 80 percent of the pipe's 50-year long-term modulus of elasticity and tensile strength are achieved. After this 1000 hour conditioning phase under a constant stress, the joint then undergoes a complete D3212 test. Any failure of this test is an indication the joint design does not have the long-term structural integrity to remain watertight. This laboratory test is a proof-of-design test of the joint and does not negate any field acceptance testing to insure the pipe was installed correctly.

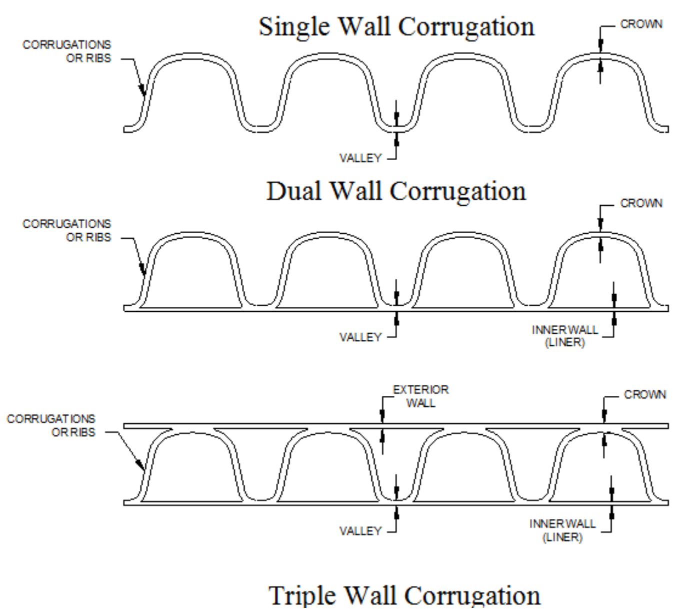

Figure 2. Standard Corrugation Configurations

ASTM F2881 covers dual wall (i.e. smooth interior and corrugated exterior wall (Figure 2)) 12-inch through 60-inch PP storm sewer pipe. These products have variable pipe stiffness which decreases with increasing diameters. The profiles and geometries of the PP pipe covered under this standard are very similar to those provided for HDPE pipe under ASTM F2306. The main difference is in the minimum pipe stiffnesses, which are on average 67% higher than those for HDPE. This higher stiffness reflects the higher modulus of elasticity for polypropylene. The material and structural requirements for F2881 are identical for those under F2736.

AASHTO M330 covers all single corrugated wall, dual wall and triple wall 12-inch through 60-inch PP storm sewer and culvert pipe (Figure 2). Pipe under this standard do not require watertight joints, and in fact, the standard includes multiple options for perforations, which would make watertight testing moot. The compound material requirements are identical to all the fore mentioned ASTM standards. As with ASTM F2881, the PP pipe under this standard has a variable pipe stiffness, which decreases with increasing diameter, but these pipe stiffnesses are 10 to 25 percent less those under ASTM F2881 depending on diameter. In part, these lower pipe stiffnesses reflect the inclusion of single corrugated wall pipe, which would have more difficulty in achieving the same values as dual and triple wall pipe due to their mass differential. The AASHTO specification does contain an additional profile wall test to assess compression capacity of the wall, the stub compression test. This compression test is intended to assess whether the extruded profile correlates with the idealized profile used to determine the structural capacity of the wall in accordance with the AASHTO LRFD Bridge Design Specifications. It is not a quality control test, but a structural evaluation test to correlate design to manufacturing.

CSA 182.13, which covers 12 through 60-inch dual and triple wall sanitary and storm sewer pipe (Figure 2), was developed after the publication of the ASTM and AASHTO standards. As such, it contains many of the same criteria as the ASTM publications. All the test protocols and requirements for the PP material and testing are identical to those in ASTM F2736 and F2764. The CSA does have a number of test protocols that are unique to Canada. Where the ASTM criterion is greater than CSA, the ASTM protocol was specified. If the two standards had the same or similar requirements, the CSA protocol was specified. There was one protocol, the watertight pressure test, where CSA exceeded the ASTM requirement, and in this case, the CSA requirement was specified. For all the watertight tests, including the 1000 hour proof-of-design joint test, a test pressure of 100 kPa (14.5 psi) is specified.

All these standards and specifications have been used for numerous sanitary and storm sewer projects in North America. Polypropylene pipe has already been approved and used by most of the U.S. state departments of transportation and the Canadian ministry of transportations, as well as numerous municipalities. As an indication of the scope and complexity of the projects that have already been installed, a small sample of these projects will be highlighted to indicate their geographic and infrastructure breadth, performance and acceptance.

PROJECT HIGHLIGHTS

The three projects that will be highlighted represent the benefits of using polypropylene pipe where cost-effective, long-term project performance was critical, and on large transportation infrastructure projects, where strength was imperative. The projects were located in Portland, Maine, Moberly, Missouri and Columbus, Ohio.

The City of Portland, in 2010, was one of the earliest users of polypropylene sewer pipe. The combined sewer separator project, designed by the engineering firm of Woodward and Curran, was a $4 million contract to eliminate overflows and keep untreated wastewater for flowing into the nearby estuary. The project scope required replacing hundred year old 24 and 30-inch vitrified clay pipe with a single larger diameter pipe, and separating a parallel, 10-foot diameter pipe to convey only stormwater. The project required the design of multiple diameters of pipe ranging from 24 to 60-inch. Dual and triple wall PP pipe were used meeting the applicable ASTM F2736 and F2764 standards. All the joints were required to be watertight.

The contractor, R.J. Grondin & Sons, had the choice of selected epoxy coated reinforced concrete pipe, centrifugally cast glass fiber reinforced polymer pipe or polypropylene pipe. From the contractor's perspective, the selection of polypropylene pipe had as much to do with ease and speed of installation, as overall watertight and structural performance. His installation costs were minimized by the ability to use smaller equipment (i.e. backhoes versus cranes (Figure 3)) and longer pipe sections that required fewer joints. The availability of custom polypropylene fabricated manhole sections also made the pipe to manhole connection process seamless.

Figure 3. Backhoe Installation of ASTM F2764 Pipe

The City of Moberly, Missouri, needed to replace or rehabilitate an old brick sewer line that had a very high infiltration rate of groundwater. The engineering firm of Jacobs Engineering determined the best option was to replace the existing line with a new 54-inch closed profile PVC pipeline or 60-inch reinforced concrete pipe. The contractor, Emery Sapp & Sons, had previously used SaniTite HP® (ASTM F2764) pipe with excellent results and proposed this pipe product as an option. Upon review, the city and engineering firm determined PP pipe would provide equal, if not superior, performance to the selected options, and allowed it as an equal alternative. The project entailed the installation of 3,300-ft of pipe with typical trench installations of 15-feet. The contractor's installation of the 60-inch PP triple wall pipe went extremely well due in part to the long lay lengths and easy joining double gasketed pipe (Figure 4), which supplied superior watertight performance.

Figure 4. Moberly 60-inch Double Gasketed ASTM F2764 Installation

The Port Columbus International Airport in Columbus, Ohio needed to add a new 10,113-ft runway to augment its expansion capacity. The project, designed by CH2M Hill, required a total of 21,580-ft of PP propylene stormwater pipe with diameters ranging from 12-inch through 60-inch. All the small diameter pipe was dual wall PP pipe (ASTM F2736) with all the larger diameters being triple wall PP pipe (ASTM F2764) (Figure 5). As with most airport runway projects, high aircraft live loads are applied to pipe with relatively shallow fills. The high stiffness PP pipe provided excellent resistance to these live loads with the added benefit of meeting high watertight integrity since both ASTM standards are for sanitary sewer pipe. Even given these higher sanitary sewer requirements, the PP dual and triple wall pipe, installed by George J. Igel & Company, was a cost effective alternative to standard RCP stormwater pipe on this project.

Figure 5. Triple Wall PP Pipe on the Port Columbus International Airport Project

CONCLUSIONS

The development of polypropylene sanitary and storm sewer pipe in North America has certainly provided unique benefits to engineers, specifiers, municipalities and contractors. For each group, it addresses a particular need. The engineers now have a thermoplastic product that is a blend between the HDPE and PVC. It provides the durability, joint integrity and strength to cost benefits of HDPE and PVC without the inherit issues of stress cracking and impact problems associated with each of these alternates, respectively. Specifiers now have another alternate to select for their installation options. The municipalities have not only another option, but a superior, cost effective alternative to both their storm sewer and sanitary sewer projects. The contractors, which have been one of the biggest early supporters of this product, see the easier installation and excellent field performance as a huge benefit to their bottom line and reputation. And in the end, isn't that what we all want?

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The Modified Use of the Rehabilitation of Water Mains Manual, AWWA M28 and ASTM F1216, to Design Large Diameter Pressure Pipes Using FRP Systems

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Abstract

As the use of Fiber Reinforced Polymers (FRP) in the rehabilitation of aging pipeline grows globally, so also does the adoption of various accepted industry ideologies of FRP design. While many municipalities in the waste water pipelines industry have used the rehabilitation of water mains standard (American Water Works Association Manual 28(AWWA M28)) and the American Society for Testing and Machines (ASTM) F1216 to develop standalone rehabilitation designs,the referenced design criteria does not account for all the loads that the pipeline may encounter in its lifetime. In addition, this criterion was not written assuming the use of an anisotropic material, such as FRP. This paper will highlight the different design practices used for FRP and Cured in Place Pipe (CIPP), as well asprovide recommendations to modify both the AWWA M28 and the ASTM F1216standard, which reflect some of typical buried pipeline load conditionsnot defined in either manual.

INTRODUCTION

Every day, millions of U.S. residents rely on water for basic needs. Yet beyond simply turning on and off the faucet, very few people think about what it takes to fill a glass of water. Today, our massive water supply system, serving millions of Americans, is in serious need of replacement or repair. The vast majority of the nation's pipelines were constructed in the 1960s or earlier and designed to last 50-100 years. Considering their constant use and age, compounded by their low rate of replacement, one could accurately conclude that most of the pipes in the U.S. are in critical need of repair or replacement.

Among the different problems plaguing buried pipes, corrosion ranks amongst the most perilous. This is caused by a natural reaction between the aggressive environment of the water and metal or the concrete substrate or the soil material/properties can cause corrosion to the host pipe from the outside. Ultimately, these issues lead to leaks, which can allow contaminants to enter the pipes water supply and consequently allows treated water to seep out of the system and be wasted. In extreme cases, eroding pipes cause the ground above them to collapse, creating sinkholes that pose a danger to the surrounding community.

In many instances, common replacement solution requires digging up city streets or highways to access the pipe, which could shut down a community's water system and create additional costs to the water authority such as repaving roads, traffic redirection, and public notifications. In response the AWWA M28 standard, originally published in 1987, was developed to provide an overview of the different technologies that could be used to rehabilitate water mains. Several of the pipeline rehabilitation methods defined in this standard include installation and design considerations for cured in place pipe (CIPP, see Figure 1), steel/HDPE slip lining, spay-on polymer lining, mortar lining, and internal joint seals. While all of these rehabilitation methods are more practical than that of traditional replacement options, there is still a significant need for an effective trenchless pipeline repair solution recognizable by owners and asset managers alike.

Figure 1. Completed Installation of CIPP

The use of anisotropic fiber reinforced polymer (FRP) composite systems in different industries, with a trenchless installation procedure, to strengthen steel and concrete pipelines has been utilized for over 20 years, see Figure 2. FRP composite systems are light weight, carbon and glass reinforcing fibers saturated with a polymer matrix. Once applied within the pipe, the system is cured within a given period of time dependent on the environmental temperature inside the host pipe. The end result is the FRP system acting as a tension member for internal pressures and in compression for external pressures. The FRP is utilized to strengthen the degraded steel/concrete by either aiding the host pipe to regain its original strength, providing additional capacity to structural elements forming the existing water infrastructure, or creating a fully structural standalone replacement pipeline. As the global structural engineering industries have adopted this technology, so have the infrastructure industries. However, there are no design standards, including AWWA M28, which refer to pipeline rehabilitation using FRP systems. Due to this ideology, the following will depict some common CIPP rehabilitation design practices used in the municipal pipeline industry, and provide a modified consensus of how to make those common design practices applicable to FRP systems.

Figure 2. Completed Installation of CFRP on a Buried Pipeline

DESIGN CONSIDERATIONS – AWWA M28

As was previously mentioned, the AWWA M28 standard was developed to provide, engineers, contractors, and other decision makers with useful rehabilitation methods for water mains. Table 1-2 in of the AWWA M28, outlines of different hydraulic improvements "available", or those specifically mentioned in the standard. One of these improvements is cured-in-place pipe lining (CIPP), but there is no mention of FRP. However hereinafter, it can be assumed the rehabilitation with FRP can be ballooned under the CIPP category. In addition, Figures 1-2 through 1-4 display different flow-charts to aid in the determination of the proper rehabilitation method. Based on these figures the use of CIPP is an appropriate choice when the host pipe has inadequate hydraulic performance, excessive leakage with few connections, trenchless, desirable, and causes low social disruption.

Moreover, Appendix A of the AWWA M28 identifies four different classifications of lining systems based on their strengthening capability when subjected to internal pressure demands. The different classifications are as follows:

- *Class I liners* addresses corrosion protection only.
- *Class II liners* address structural deficiencies for gaps and holes in the pipe.
- *Class III liners* address structural deficiencies for gaps and holes, as well as external loads.
- *Class IV liners* act as a pipe within a pipe taking all internal and external loads acting on the pipeline system.

The discussion on CIPP is continued by outlining several benefits of using a plastic material to rehabilitate a water main. The first is due to the installation process, the interior of the pipe will be extensively cleaned, which will help restore the host pipe to its original design dimensions. It was also noted that liners utilizing CIPP/FRP may have a smoother interior surface once the material has cured, which can reduce roughness coefficients and increase the flow rate within the pipe making it more efficient. The coefficient of friction value (C values) for FRP is considered to be relatively low; similar to that of PVC. While test results do not exist for C values for FRP-lined pipe, it can be considered similar to that of an epoxy coated steel pipe or a fiberglass pipe for which AWWA recommends a C value of 150. Finally, since

CIPP/FRP is essentially joint-free coverage over the extents of its application, this can offer fewer disturbances to the water flow than if the pipe is repaired with jointed sections.

In regards to projects in the field, when owners, engineers and operators typically request the CIPP (and consequently FRP) repair method, the primary assumption is the repair will fall under the Class IV lining system. This implies the strengthening solution will be designed as a stand-alone system and encompass two important characteristics:

1. The solution is a long-term solution (at least 50 years) that can withstand the internal burst pressure demands currently being placed on the rehabilitated pipeline.
2. The design can also withstand any dynamic loading (or other short-term loading) due to a sudden and complete failure of the host pipe due to and transient and vacuum internal burst pressure demands.

Also, the design requirements for a Class IV lining system should be the same as the host pipe, which implies several other design considerations (in addition to the two mentioned above) are required, i.e. external buckling loads, longitudinal/bending strength, and traffic live loads. Unfortunately, this is the extent of the direction given in the AWWA M28 standard for the design of a Class IV lining system.

DESIGN CONSIDERATIONS – ASTM F1216

Now imagine a thorough review of the host pipe has been performed and it has been determined that a Class IV lining system is required to rehabilitate the pipe … What is the next step? Unfortunately, there is no clear answer to this question within the AWWA M28 standard. Appendix A gives some clue of the varying parameters, which should be considered in the design, but there is no reference to any design standards that would aid in the design of the CIPP/FRP system.

This implies the design engineer now needs to look to other design standards to determine the CIPP/FRP design solution. Fortunately, there are several design standards that have been published over the last decade to determine the appropriate design. To design for the following classifications listed in AWWA M28, many municipalities in the wastewater and potable water industries have adopted the use of the American Society for Testing and Materials (ASTM) developed standard entitled *"Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin-Impregnated Tube (F1216)"*, which was specifically developed for the design of CIPP and directly references the AWWA M28 Manual. Initially published in 1998, ASTM F1216 provided design engineers a set of minimum design parameters based on testing performed on pipes strengthened with CIPP. The basis of the design was to evaluate the minimum design thickness required to achieve a Class II/Class III (partially deteriorated) or Class IV (fully deteriorated) repair by looking at seven different design equations. Table 1 outlines the equations required for a given rehabilitation classification.

This standard is only for the application of CIPP in the circumferential direction (considering CIPP is an isotropic material), however based on the design equations, one could consider using FRP values to determine a minimum thickness that would satisfy either the Class II/Class III or Class IV repairs. For fully deteriorated repairs (Class IV), the original pipe is assumed not to be structurally sound or be able to support any soil/live loads. This consideration is likely to occur if the original pipe is missing, has lost its original shape, or is corroded due to the effects of the fluid atmosphere, soil, or applied loads. In the following sub-sections, the proper design equations are presented, which are required for a Class IV structural repair for pressure (and water main) pipelines.

Table 1. Equations to be satisfied based on System Type

System Type	***Equations***
Partially Deteriorated	X1.1, X1.2, X1.6
Fully Deteriorated	X1.1, X1.2, X1.3, X1.4, X1.7

Equation X1.1- Groundwater Load Check

This design equation requires that the CIPP/FRP system withstand the hydraulic loads due to groundwater. In addition, based on the fully deteriorated pipe assumption, it is required to have both the soil and surcharge loads included in the design equation. However, it is important to note that this equation was developed assuming the host pipe was not completely deteriorated. This implies that the existing pipe and surrounding soil will contribute to the overall stiffness of the CIPP/CFRP System; hence there is an enhancement factor, K in the equation below, to account for this. The inclusion of this factor has led to discussions over the validity of this equation for fully deteriorated pipes.

In the end, the equation below satisfies the groundwater (plus the additional soil and surcharge) load requirement:

$$P_{\substack{FRP \\ CIPP}} = \frac{2K(f_t E_T)}{(1-\nu^2)} \frac{1}{(DR-1)^3} \frac{C}{N} > P_{ground}$$

where P_{ground} is the groundwater, soil and surcharge loads (psi), DR is the dimension ratio for the FRP/CIPP system, K is the enhancement factor of the soil and existing pipe (minimum value of 7 is suggested), f_t is the reduction factor for complete design life of system, E_T is the flexural elastic modulus of the FRP/CIPP system (psi), ν is the Poisson's Ratio (typically 0.3, if not known), C is the ovality factor (defined on pg. 7 of ASTM F1216), and N is the assumed factor of safety.

Equation X1.2- Pipe Ovality Check

The ovality (or roundness) of the strengthened pipe must satisfy the following equation:

$$1.5\frac{\Delta}{100}\left(1+\frac{\Delta}{100}\right)(DR)^2 - 0.5\left(1+\frac{\Delta}{100}\right)(DR) > \frac{f_f \sigma_F}{P_{ground} N}$$

where Δ is the percent ovality of the original pipe, DR is the dimension ratio for the FRP/CIPP system, f_f is the long-term flexural strength reduction factor, σ_F is the flexural strength of the FRP/CIPP system (psi), P_{ground} is the groundwater, soil and surcharge load (psi), and N is the assumed factor of safety.

Equation X1.3- Buckling Load Check

For a fully deteriorated pipe (i.e. no additional support from the host pipe), the design of the FRP/CIPP system is required to support hydraulic, soil, and live loads and must satisfy the equation below:

$$q_{\substack{FRP \\ CIPP}} = \frac{1}{N}\left[32R_w B' E_s' C\left(\frac{E_T I}{D^3}\right)\right]^{1/2} > q_{tdemand} = 0.433H_w + \frac{wHR_w}{144} + W_s$$

where N is the assumed factor of safety, R_w is the water buoyancy factor (defined on Pg. 8 of ASTM F1216), $B'$is the coefficient of elastic support, E_s' is the modulus of soil reaction (psi), C is the ovality factor, E_T is the elastic modulus of the FRP/CIPP system (psi), I is the moment of inertia of FRP/CIPP system (in^3), H_w is the height of water above the pipe (ft), w is the surrounding soil density (pcf), H is the height of soil above the pipe (ft), and W_s is the calculated live load on pipe (psi).

Equation X1.4- Soil Modulus Reaction Check

The equation below states the minimum design requirements due to the strength of the surrounding soil:

$$\frac{E'_s I}{D^3} = \frac{E'_s}{12(DR)^3} \geq 0.093\ (inch - pound\ units)$$

where E_s' is the modulus of soil reaction (psi), I is the moment of inertia of FRP/CIPP system (in^3), D is the mean internal diameter of the host pipe (in), and DR is the dimension ratio for the FRP/CIPP system.

Equation X1.7- Internal Pressure Design

The final design equation below determines the minimum required thickness for the FRP/CIPP system, in order to withstand the full internal pressure demand:

$$P_{\substack{FRP \\ CIPP}} = \frac{2\sigma_T f_t}{(DR-2)N} > P_{demand}$$

where f_t is the long-term tensile strength reduction factor, σ_T is the tensile strength of the CIPP system (psi) or the tensile elastic modulus of FRP, DR is the dimension ratio for the FRP/CIPP system, N is the assumed factor of safety, and P_{demand} is the internal pressure demand on the rehabilitated pipe (psi)(i.e. working and transient pressures).

Upon further review of the equations mentioned above, it is important to note that these design equations look independently at the design parameters. For instance, the internal pressure and external loading demands are checked with two separate equations, but are never assumed to be acting simultaneously. In addition, there are several other drawbacks of the ASTM F1216 for FRP systems: (1) the exclusion of the design requirements for the longitudinal direction, which is necessary for anisotropic materials; (2) use of the soil enhancement factor and factors of safety without proper direction of implementation; and (3) the long-term reduction factors for flexure and tension are assumed by the designer without limitations.

POSSIBLE DESIGN STANDARDS FOR FRP SYSTEMS

Since its first application in 1997, the use of CFRP for internal repair and strengthening of PCCP or steel has slowly been accepted within the utilities industries. Considering the Class IV pipeline repair is a full structural repair, or simply creating a pipe within a pipe, some utilities have implemented the same design standards used for steel or PCCP pipe. With the only difference being that the design equations are modified to include the FRP design values in place of the steel/PCCP design values.

One such standard is the AWWA M11: "*Steel Water Pipe – A Guide for Design and Installation*", which describes the design procedure to determine the required thickness for a full structural design. By modifying the equations, the number of layers (in both the circumferential and longitudinal directions) can directly be determined. This design approach examines different design criteria based on the loading conditions on the pipe, i.e. earth, live, internal static pressure, internal surge pressure, and the self-weight of the host pipe. This design procedure also gives clear direction for the design of thrust forces in the longitudinal direction, due to changes in the direction of pipe, i.e. bends. However, it is important to note that these design equations are only applicable for steel pipelines and caution should be taken when applying them to other pipe materials, especially when examining the deflection and buckling equations.

Currently, the AWWA has been developing a standard that specifically outlines the strengthening/rehabilitation design requirements of Prestressed Concrete Cylinder Pipes (PCCP) when utilizing FRP (AWWA Draft). This design standard not only investigates the basic principles of buried pipe design, i.e. pressure, buckling, deflection, but also takes into consideration the inherent material properties of FRP. The creation of the standard was developed primarily from three existing AWWA manuals: (1) AWWA C304: *"Standard for Design of Pre-Stressed Concrete Cylinder Pipe"*, (2) AWWA M9: *"Concrete Pressure Pipe"*, and (3) AWWA M45: *"Fiberglass Pipe Design Manual"*. In the end, this new design standard addresses the anisotropic properties of FRP systems, which implies that the application of FRP systems is required not only in the circumferential direction, but also in the longitudinal direction. Nonetheless, one of the true benefits of this design standard for FRP systems is its focus not only on the material itself, but also on the quality control and installation of the material. Considering the installation of the FRP system is directly related to the performance of the material, this standard also includes requirements on the designer, manufacturer and installer of the FRP system, which should also be included in the project specifications. This will ensure the FRP system is not only designed properly but installed to maintain its performance requirements per the design standard. In addition, the AWWA Draft Standard includes the design requirements for composite design, i.e. the host pipe has strength that can be included.

Ultimately, Table 2 compares the differences between the various design standards mentioned above. It is important to note, that not all the design standards will fit a project perfectly. There will be situations, where a combination of different standards will be required. This implies there is a great deal of responsibility placed on the design engineers shoulders to review, evaluate, and decide which standards will be best fit a specific project's needs. There may never be a situation where a plug and chug approach will garner a cost effective, reasonable design solution. That being said, the following section provides suggested modifications to the AWWA M28 and ASTM F1216 when designing primarily for FRP systems, with considerable overlap in the design of CIPP systems as well.

Table 2. Comparison of Design Standards

	M28	**F1216**	**M11**	**PCCP Draft**
General Design Requirements				
- Host Pipe Material	Varies	Varies	Steel	PCCP
- FRP/CIPP Design Standard	X	X		X
- FRP/CIPP Material Property Req'ts		X		X
- FRP/CIPP Installation Procedures	X			X
- Quality Control/Assurance				X
Circumferential Design	X	X	X	X
- Internal Pressure Demands	X	X	X	X
- External Loading Demands	X	X	X	X
- Combined Internal/External Loading			X	X
- Buckling Limitations	X		X	X
- Deflection Limits		X	X	X
Longitudinal Design	X		X	X
- Poisson's Effect			X	X
- Thrust Loading			X	X
- Temperature Effects				X
- Bending due to Radial Effects	X			X

*FG = Fiberglass

MODIFICATIONS AND INTEGRATION OF RECOMMENDED FRP STANDARDS TO AWWA M28 AND ASTM F1216

Unfortunately, neither the AWWA M28 nor the ASTM F1216 gives the clearest of design procedures for FRP systems. This being said, there are several ways the AWWA M28 could be modified in order to reduce confusion and instead provide consistent and reliable design solutions for buried water mains, which are:

1. References to other standards should be included. This will provide guidance to the design engineer, in order to determine the proper standard for their given project.

2. Considering FRP is a viable solution to many water main rehabilitation projects, the inclusion of FRP as a rehabilitation option should be included. This requires either the addition of a chapter or a combined CIPP/FRP chapter, but would allow design engineers the ability to understand the inherent benefits of the FRP solution. Also, installation and quality control/assurance requirements should not be excluded.
3. Modification of Appendix A to include all the necessary design parameters for the design of the CIPP/FRP systems in a clear and concise manner. This implies including the references to standards (see 1 above), as well as clearly stating the different design states that should be analyzed. These include, but are not limited to:
   a. Requiring a deflection limit in the circumferential direction.
   b. Effects of combined internal/external loading in the circumferential direction.
   c. Examining longitudinal burst pressure. This includes looking at Poisson's effect, thrust loading due to bends, and the effects of temperature change during/after installation.

While for the ASTM F1216, suggested modifications refer not only to design requirements, but also installation practices for the FRP system. The design modifications include:

1. The addition of design equations specifically for anisotropic materials, i.e. design equations in the longitudinal direction (similar to 3c above). This includes looking at Poisson's effect, thrust loading due to bends, and the effects of temperature change during/after installation. A suggested design procedure is outlined in the three-step process below:

   STEP 1: *Determine the minimum number of layers required using Poisson's Effect.*

   $$n_L = \frac{v_{FRP} n_H t_{f.h}}{t_{f.L}}$$

   where n_L is the minimum number of layers required in the longitudinal direction, v_{FRP} is the Poisson's Ratio, n_H is the minimum number of layers required in the circumferential direction, $t_{f.h}$ is the thickness of a single layer of the FRP system in the circumferential direction (in), and $t_{f.L}$ is the thickness of a single layer of the FRP system in the longitudinal direction (in).

   STEP 2: *Use the number of layers calculated in Step 1 to determine if the design meets the design demands due to Thrust Loading (only required if bends are present).*

   $$T_{DEM} = T_{red} A_{pipe} P_{demand} \leq f_t \varepsilon_{ALL} E_{T.L} A_L = T_{ALL}$$

   where T_{red} is the thrust reduction factor equal to $(1 - \cos\theta)$, θ is assumed bend angle, A_{pipe} is the area inside the pipe (in^2), P_{demand} is the internal pressure demand (psi), f_t is the long-term tensile strength reduction factor, ε_{ALL} is the allowable design strain for the FRP in the longitudinal direction (in/in), $E_{T.L}$ is the elastic modulus of the FRP system in the longitudinal direction (psi), and A_L is the cross-section area of the longitudinal FRP (in^2).

STEP 3: *Use the number of layers calculated in Step 1 to determine if the design meets the demands due to Temperature Effects.*

$$\varepsilon_{DEM} = \left(\frac{\alpha_{11} A_L E_{T.L} + \alpha_{22} A_H E_T}{A_L E_{T.L} + A_H E_T}\right) \Delta T \leq f_t \varepsilon_{ALL}$$

where α_{11} is the coefficient of thermal expansion in the fiber direction (/°F), A_L is the cross-section area of the longitudinal FRP (in^2), $E_{T.L}$ is the elastic modulus of the FRP system in the longitudinal direction (psi), α_{22} is the coefficient of thermal expansion in the transverse direction (/°F), A_H is the cross-section area of the circumferential FRP (in^2), E_T is the elastic modulus of the FRP system in the circumferential direction (psi), ΔT is the assumed temperature change in the pipe (°F), f_t is the long-term tensile strength reduction factor, and ε_{ALL} is the allowable design strain for the FRP in the longitudinal direction (in/in).

2. Clarification of the proper method to determine the surcharge or live loads. It is recommended that Table 4.1-1 (Page 13) in the American Lifelines Alliance document entitled *"Guidelines for the Design of Buried Steel Pipe"* be used to determine these loads, with an additional statement included within the project specifications.

While the additional modifications below are additional statements required for proper installation practices:

1. If the deteriorated pipe is a steel pipe, it is necessary for a dielectric barrier to be installed prior to the carbon FRP system, in order to prevent potential galvanic corrosion. This is only required if the liner is removed and the substrate that the carbon FRP system is going to bond directly to is steel.
2. In order for the FRP system to have an adequate bond to the host pipe, it is necessary for the host pipe to always have proper surface preparation. The surface preparation can be checked by including a bond strength requirement of 200 psi into the project specifications. It is suggested to prepare concrete surfaces such that the concrete aggregate is exposed or Concrete Surface Profile-3 (CSP-3) as defined by the International Concrete Repair Institute Industry guidelines (ICRI). It is suggested that steel pipeline surfaces be prepared to near white metal as defined in the Society for Protective Coatings Surface Profile 10 (SSPC SP-10).

CONCLUSIONS

In the end, the primary focus of this paper was on the AWWA M28 and the ASTM F1216 standards and their inherent flaws when designing a stand-alone rehabilitation solution using either FRP or CIPP systems. In general, the purpose of the AWWA M28 standard was to provide engineers, contractors, and other decision makers with general process, which is useful to determine the proper water main rehabilitation. As with other documents of a similar nature, the AWWA M28 is only an overview. This implies that there is not always a clear direction for the development and implementation of the design. Based on project experience, it has been determined that modifications to the AWWA M28 are required to ensure consistent and reliable rehabilitation solutions are created if CIPP/FRP system is the design solution. It was concluded that three inclusions were necessary: (1) references to other standards should be included to aid in the design process, (2) include FRP as a viable design solution, and (3) modify Appendix A to include all the necessary design parameters and not just a few.

While for the ASTM F1216 was initially developed as a design standard for CIPP systems. Upon further review of the standard, it was determined that additional design modifications and installation practices are necessary when designing for FRP systems. Considering the anisotropic nature of the FRP systems, it is important to include a design procedure not only in the circumferential direction, but also in the longitudinal direction. The suggested design equations/procedure, for the longitudinal direction, was influenced by the design requirements in both the AWWA M11 and the AWWA Draft Standard. Nevertheless, it is also important to consider the installation of the FRP system since this directly relates to the strength and reliability of the system. This implies that it is necessary to include statements on the proper installation of the FRP system to the host pipe. For the purposes of the ASTM F1216, only two additional modifications are suggested: the requirement of a dielectric barrier to prevent galvanic corrosion when installing carbon FRP systems and the necessity for the host pipe to always have proper surface preparation prior to installation of the FRP system.

Nevertheless, it is important to understand the differences between the different design standards presented in this paper. Table 2 displays those differences and allows a designer to visually understand the varying aspects mentioned in each standard. Unfortunately, there is not a consistent design standard used in the FRP/CIPP rehabilitation/strengthening industry. Consequently this allows for any given project, a variation of design assumptions, requirements and solutions. By understanding the intrinsic qualities of each standard described in the above paper as well as utilizing steel and FRP strain compatibility instead of ultimate FRP strains, it will allow a designer to be well informed and their design standard choice would be logical for any given project’s needs and requirements. Please note that AWWA is actively working with committees in the FRP industry to adopt alternate repair options using FRP in there standards.

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Why Design Engineers Do Not Follow AWWA M9 Chapter 9? Here Are Some Suggestions to Encourage Its Use

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Abstract

The author was retained by the DC Clean Rivers Project team while designing the relocation of a critical water main of PCCP with bends to minimize the adverse effects from the construction of a 75-foot in diameter and 100-foot deep shaft. In this project, the client sought multiple pipe materials considered for the new bends. The author used most of the thrust restraint design methods presented in the AWWA manuals of practice with ease, but stumbled into many issues in Chapter 9 of M9. The author examined:

- The details of the new thrust restraint design method
- The content of Zarghamee et al. (2004), as well as the discussions and the closure, Zarghamee et al. (2005) that were written in response to this paper
- The experimental verification of the new design method by ACPPA and the AWWA concrete pressure pipe committee before implementing Zarghamee et al. (2004) in Chapter 9 of AWWA M9

This paper summarizes the lessons learned from the birth and the life of Chapter 9 in M9 and the associated software TRDP. This paper's most significant contribution to the profession is outlining changes that would encourage wider use of the contents of AWWA M9 Chapter 9.

INTRODUCTION

During late 1988 to early 1997, there were nine failures in the Richland pipe line composed of PCCP owned and operated by the Tarrant Regional Water District (TRWD) reported by Marshall (1998). The causes of failure were attributed to thrust (4), operator error (1) and corrosion (4) by the owner, the engineer, Freese and Nichols, and the pipe manufacturer, Gifford Hill (now Hanson Pipe and Products). TRWD, the pipe manufacturer, and the engineer retained the engineering firm Simpson, Gumpertz and Heger (SGH) as a consultant. According to Marshall (1998), SGH concluded that, based on finite element analyses (FEA), highly plastic soils did not offer sufficient side support, so they redesigned the pipe adjacent to the bends with thicker cylinders that could resist tensile forces associated with the movement. These four failures, for which lack of sufficient thrust restraint was blamed by SGH, happened about 25 years ago. To date, no other water utility has experienced similar thrust-induced failures that have been related in the same manner to restrained joint calculation procedures anywhere in North America on concrete pipelines.

The American Concrete Pressure Pipe Association (ACPPA) took more than a decade updating the thrust restraint design to publish the third edition of AWWA M9 in

2008. The basis of the new Chapter 9, which deals with thrust restraint, was first published in Zarghamee et al. (2004). Discussions by three peers and the closure by the authors appeared in September 2005. ACPPA subsequently funded the development of software to implement the new extremely complex thrust restraint design method. Soon thereafter, in 2009, ACPPA began to distribute computer software named Thrust Restraint Design Program (TRDP) on a thumb memory stick. This was free of charge and a year later, it also became freely available for download from the website hosted by the ACPPA.

The author has come across only one engineer — with graduate degrees in geotechnical and structural engineering, and licenses in numerous states — who attempted to understand completely the engineering principles in the new method; however, he did not succeed because of the mathematical complexity of the approach and the lack of many intermediate steps needed to use an engineering design manual properly. Only a few engineers have used TRDP for designing projects for their clients. Within this group, not many have taken the time needed to even follow the three design examples in M9 properly, due to essential intermediate steps missing.

ACPPA and Zarghamee et al. (2004) need to be commended for their effort in developing the new design tool. There are shortcomings, however, which fall into four types:

- Improper assumptions and the execution of widely accepted principles and practices of geotechnical and structural engineering failing to meet the legal definition of "standard of care"
- Unwarranted complexity yielding insignificant improvement of the state of the art
- Layers of conservatism that suffocate the noble goal of achieving a more efficient use of engineering materials in the design and manufacture of concrete pressure pipe in coping with unrestrained thrust
- Failure to include "how to" intermediate steps and poor documentation

The "standard of care" is defined as the watchfulness, attention, caution and prudence that a reasonable person in the circumstances would exercise. If a person's actions do not meet this standard of care, then his/her acts fail to meet the duty of care that all people, supposedly, have toward others. Failure to meet the standard can be considered negligence, and any damages resulting may be claimed in a lawsuit by the injured party. The engineer who uses methods or software that he is unable to verify or understand assumes the risk of not meeting the standard of care.

AWWA M9 CHAPTER 9 NOT THE SAME AS ZARGHAMEE ET AL. (2004)

As presented, the new method appears to use sound engineering principles of soil-pipe interaction. Unfortunately, this is neither the case in the assumptions made nor in the rigor that embodied the formulation of the attributes that are present under field conditions. For example, Zarghamee et al. (2004) reads, "*The thrust at a buried horizontal bend is resisted by the frictional resistance of the soil against axial* *movement of the pipe as well as the passive resistance of the soil to the transverse movement of the pipe.*" To this day, no efforts have been made to represent the forces at work correctly to satisfy force equilibrium. Zarghamee et al. (2004) also reads that, "*The frictional resistance of the soil against axial movement of the pipe gives rise to an axial force, and the passive resistance of the soil to the transverse movement of the pipe gives rise to shear and bending in the pipe.*" This is so only if the true axial force and the shear in the pipe satisfy both equilibrium of forces and compatibility of deformations. The true axial force, however, can be calculated only if the true magnitudes of the resisting forces from the passive soil resistance and those from soil-pipe interface friction are considered, without ignoring any portions of these forces. Unfortunately, terms that belong to the true passive resistance and frictional resistance were incorrectly calculated in Zarghamee et al. (2004) by not using the correct geometry or dimensions of all possible designs for bends, and by not using the direction in which the movement of the bend into the soil would take place in response to the unbalanced thrust resultant.

To develop the new method, Zarghamee et al. (2004) performed a series of finite element analyses of the behavior of concrete pipe near the bend and proposed a design procedure with welded joints for the equilibrium of forces shown in Figure 1.

$$T = 2PA\sin\left(\frac{\Delta}{2}\right) = 2F_o\sin\left(\frac{\Delta}{2}\right) + 2V_o\cos\left(\frac{\Delta}{2}\right) + 2k\delta l'_b\cos\left(\frac{\Delta}{2}\right) + 2f\mu l_b\sin\left(\frac{\Delta}{2}\right) \quad (1)$$

With T= unbalanced thrust, lb
P= internal pressure, psi
A= cross-sectional area of the pipe joint, in.^{2}
Δ = deflection angle of the bend, in degrees
F_o= axial force in the pipe at the fitting, lb
V_o= shear in the pipe at the fitting, lb
l'_b= length of leading edge of fitting from the joint, in f_μ
l_b= centerline length of fitting joint from the point of intersection, in
δ = outward movement of fitting, in.
k = soil stiffness against outward movement of the pipe or fitting, lb/in./in.

Unfortunately, Zarghamee et al. (2004) did not use the correct terms to represent the resistance from the soil in equation 1; passive earth pressures and soil-pipe interface friction are based on inappropriate assumptions about the way the bend behaves when it interacts with the surrounding soils and in the estimation of the areas over which such resistance forces are at work. Zarghamee et al. (2004) correctly suggested fastening a number of pipe joints on each side increases the frictional drag of the connected pipe and resists the thrust acting on the fitting used for the bend, but they did not calculate the frictional drag properly. The frictional resistance, f_μ, of a buried pipe, is expressed in AWWA M9 as

$$f_\mu = \mu\,[(1 + \beta)W_e + W_p + W_f] \quad (2)$$

With μ = coefficient of friction between pipe and soil
W_e = earth load, lb./ft.
W_p = weight of pipe, lb./ft.
W_f = weight of fluid in pipe, lb./ft.

β = shallow cover factor

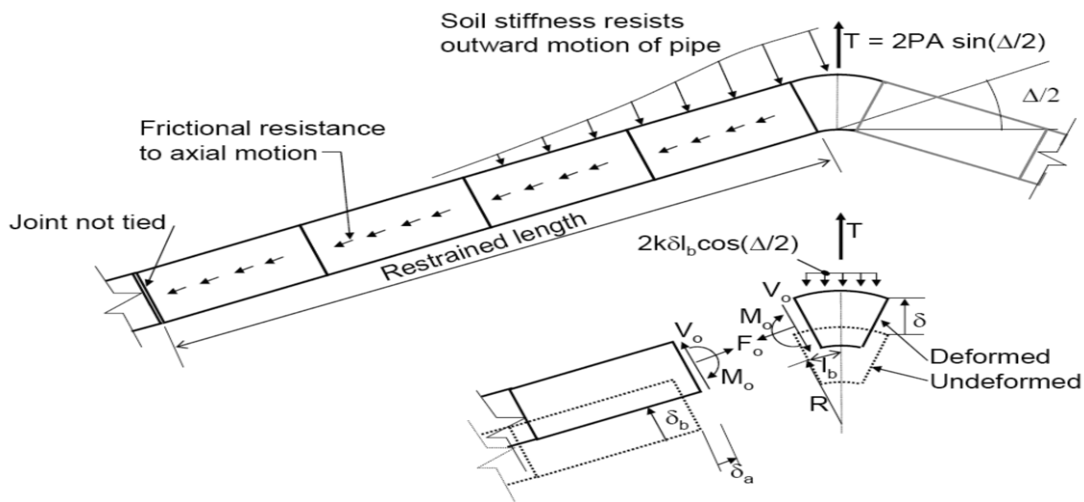

Figure 1. Free body diagram of a bend (AWWA, 2008)

Zarghamee et al. (2004) felt that, when the pipe has a shallow cover, the soil on the top of the pipe may move with the pipe. Soil resistance against movement of the pipe is provided, in part, along the sides of the soil block directly above the pipe, rather than at the pipe-to-soil interface along the top surface of the pipe. Hence, the shallow cover factor, β, was expressed by the following:

$$\beta = \frac{K_o \tan\phi}{\mu} \frac{\left(\frac{12H}{D_o}+0.5\right)^2}{\left(\frac{12H}{D_o}+0.107\right)} \leq 1 \quad (3)$$

With K_o = 1 – sin ϕ, Jaky's lateral earth pressure coefficient at rest
ϕ = angle of internal friction, in degrees
H = depth of cover, ft.
D_o = outside diameter of pipe, in.

Zarghamee et al. (2004) also included the weight of the soil placed in the upper haunches, which is ignored in all other AWWA Manuals of Practice, M11, 23, 41, 45 and 55. These consider only the weight of a purely rectangular prism of soil bounded by the horizontal plane at the top of the pipe, the vertical planes at the sides and the ground level. Given that the prism earth loading concept is based on an approximation to begin with, and that the weight of this small volume of soil is small compared with the rectangular prism of soil above the pipe; therefore, the added weight is negligible compared to the heavy concrete pipe, especially when compared to the relatively light pipe materials steel, PVC, ductile, fiber glass and HDPE. Thus, the inclusion of the upper haunch soil weight is unfounded and adds needless complexity.

The author examined the impact of the shallow cover correction factor, β given in Chapter 9 of M9 and found that the lowest value it ever takes is 0.9 and for only less than 5 % of the cases, as shown in Figure 2. When combined with the offsetting impact of refining the estimate for the weight of backfill by including the soils in the upper haunches, the net effect becomes most insignificant in Figure 3.

The displacements of the pipe are shown correctly in Figure 1. Compatibility of displacements is ensured by expressing the axial deformation of the pipe, δ_a (in.), and the transverse deformation of the pipe, δ_b (in.), in terms of the outward movement of the fittings, δ, as follows:

$$\delta_a = \delta \sin\left(\frac{\Delta}{2}\right) = \frac{12F_o L_{ft}}{2E_c A_t} = \frac{6F_o L_{ft}}{E_c A_t} \tag{4}$$

$$\delta_b = \delta \cos\left(\frac{\Delta}{2}\right) = \frac{F_{o\lambda}}{k} \tag{5}$$

With D_y = steel cylinder outside diameter, in.
E_c = modulus of elasticity of concrete, psi
A_t = transformed area of the pipe wall cross section = $A_c + nA_y$, in.^{2}
A_c = $\frac{\pi}{4}\left(D_o{}^2 - ID^2\right) - t_y\pi\left(D_y - t_y\right)$, in.^{2}
A_y = $\pi\left(D_y - t_y\right)\left(t_y\right)$, in.^{2}
λ = beam on elastic foundation parameter
L_{ft} = length of restrained pipe, ft.
n = modular ratio of steel to concrete
ID = inside diameter of pipe, in.
D_o = outside diameter of pipe, in.
t_y = thickness of steel cylinder, in.

The value of λ is expressed by

$$\lambda = \left(\frac{k}{4E_c I_{eff}}\right)^{1/4} \tag{6}$$

With I_{eff} = $\left[\frac{\pi}{64}\left(D_o{}^4 - ID^4\right) - I_s\right]\psi + nI_s$ with $\psi = 0.2$
= effective moment of inertia of the pipe wall cross section, in.^{4}
I_s = moment of inertia of the steel cylinder, in.^{4}

The restrained length of pipe, L_{ft} (ft.), measured from the bend, is calculated as

$$L_{ft} = \frac{F_o}{f_u} \tag{7}$$

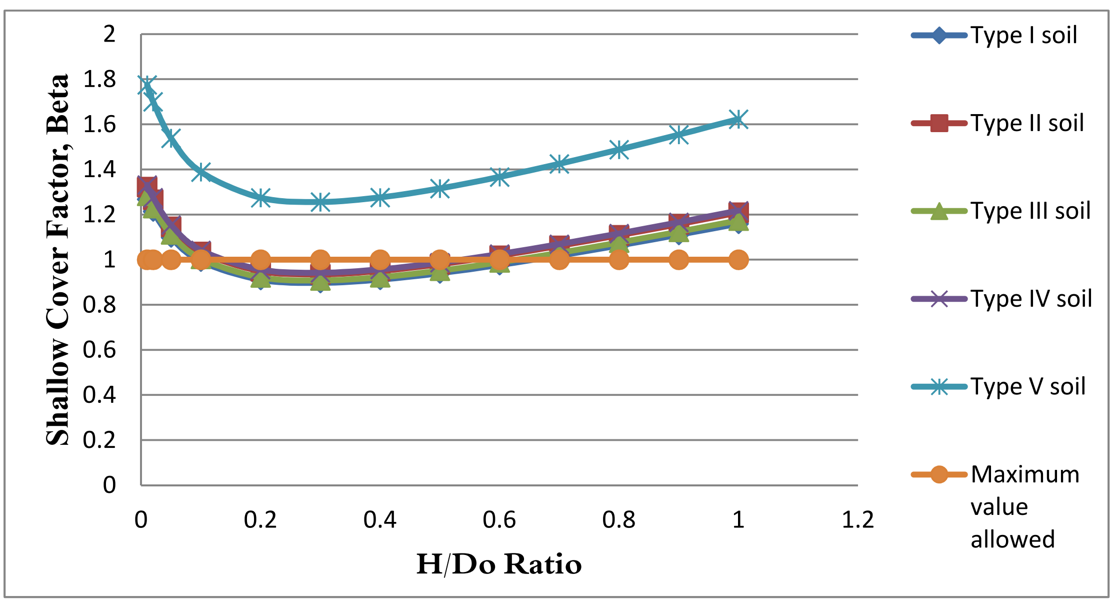

Figure 2. Effects of shallow cover correction factor, beta only

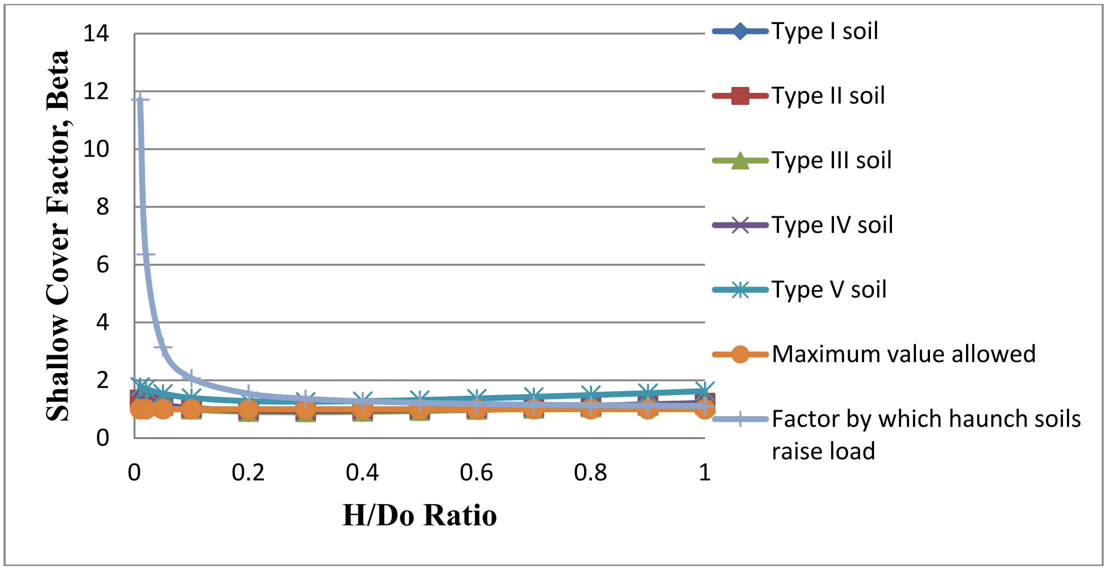

Figure 3. Effects of shallow cover correction factor and upper haunch soils

The soil Types I, II, III, IV, and V are defined in Table 9-1 of AWWA (2008). Equations 1, 4, 5 and 7 are solved simultaneously to estimate F_o, V_o, L_{ft}, and δ. In fact, for most cases, this amounts to solving a simple quadratic equation for one variable. The bending moment M is calculated from shear V_o using the beam on elastic foundation equation, $M = \frac{V_o}{2\lambda}\left[e^{-\lambda x}(\cos\lambda x - \sin\lambda x)\right]$. These values are used correctly to determine the stress resultants along the pipeline with welded joints.

DISCUSSIONS IN 2005 AND 2013

There were discussions written on Zarghamee et al. (2004) and the most credible and thoroughly researched was by Dana (2005), who wrote, "*The authors calculate the effective moment of inertia of pipe by multiplying the transformed moment of inertia, neglecting mortar coating, of an uncracked pipe by a factor of ψ = 0.2. Although the authors report that their moment of inertia matches results from their finite-element model for a 72 inch (1.829 m) diameter pipe, it is less than half the cracked moment of inertia. The authors' serviceability criteria allow only the onset of microcracking, but it is unlikely that the moment of inertia could be reduced to the authors' value by joint flexibility and tensile softening of concrete without severe cracking. The small moment of inertia proposed by the authors raises concerns about the validity of their proposed design procedure and the accuracy of the FEA in simulating actual field performance"* Nevertheless, the use of the correction factor $\psi = 0.2$ was added to the third edition of M9.

Although the author was unable to establish direct dialogue by phone with the senior writer of Zarghamee et al. (2004), the author exchanged emails with Zarghamee (2013), from September to December, 2013, while a few members of the AWWA Concrete pressure pipe committee were witnesses. The author asked, "*Although it is expected that the moment of inertia from M9 page 153 should always come out to be* *larger than that from M9 page 175, even to match these two, I had to use ψ values ranging from 0.05365 to 0.33 in the five cases on the same DC Clean Rivers Project and in design examples 2 and 3 in Chapter 9 of M9 (see Table 1a); this implies that, the current formulation for I_{eff} requires a correction from 95 to 67% signaling that the correction is too large, thereby undermining its credibility. What evidence do you cite to support your assertion that this is not an issue?"* The senior writer of Zarghamee et al. (2004) replied *"See ACI 318-11, Chapter 10. I draw you (sic) attention to Sec. 10.10.4 and Sec. 10.10.6.1 Equation (10-14). Considering 20% of concrete stiffness for a member that is subjected to bending moment and tension is reasonable as concrete softens by microcracking and cracking."*

Table 1a. Analysis of Moments of Inertia of Concrete Pressure Pipes

	C301	C301	C301	C301	C300	M9design	M9design
pipe type	ECP	ECPClC	LCPSP5	LCPS5	RCCP	example2	example3
nominal inside dia(in.)	48	108	36	36	96	54	42
pipe outside dia(in.)	65	123.125	42.394	42.2926	113	58.375	51
core thickness(in.)	7	6.75	2.25	2.25	8.5	0.9375	3.5
core thick outside the cyl(in.)	5.625	4.5	0	0	4.7362	0	1.25
core outside dia(in.)	62	121.5	40.5	40.5	113	55.875	49
mortar coating(in.)	1.5	0.8125	0.8125	0.8125	0	1.25	1
outer core & coat thick(in.)	7.125	5.3125	0.8125	0.8125	4.7362	1.25	2.25
cylinder nominal dia(in.)	50.75	112.5	40.5	40.5	103.5276	55.875	44.5
cylinder outside dia(in.)	50.75	112.5	40.50	40.5	103.5276	55.875	44.5
cylinder thickness(in.)	0.0598	0.0598	0.1345	0.0838	0.50047	0.1875	0.1046
concrete strength(psi), fc	4800	4800	5250	5250	4500	4500	4500
mod of elast of conc(ksi)	3949	3949	4130	4130	3824	3616	3616
conc tensile strength(psi)	485	485	507	507	470	470	470
unit weight(pcf)	145	145	145	145	145	145	145
steel cyl yield strength(psi)	27000	27000	27000	27000	27000	27000	27000
mod of elast of steel(ksi)	30000	30000	30000	30000	30000	30000	30000
modular ratio	7.6	7.6	7.3	7.3	7.8	8.3	8.3
Moment of I- ψ = 0.2 (in.^{4})	**145816**	**1168085**	**39902**	**30357**	**2401288**	**133572**	**65008**
Moment of I – Gross (in.^{4})	**615912**	**4604757**	**76140**	**74628**	**3835892**	**152672**	**179413**
Moment of I – Transf (in.^{4})	**636097**	**4825174**	**98035**	**88321**	**5297990**	**245533**	**205662**
Moment of I – Crack (in.^{4})	**56115**	**601937**	**44097**	**30537**	**2715001**	**152897**	**58155**

NOTE: The moments of inertia with values underlined are cases in which $\psi = 0.2$ yield lower than those for even the lowest moments of inertia corresponding to cracked sections. This indicates that the use of ψ = 0.2 needs further work.

The author wrote back, *"ACI 318 -11 section 10.10 deals with 'Slenderness effects in compression members.' I ask, 'What is the relationship between this section 10.10 and the calculation of the effective moment of inertia of the pipe cross section that is undergoing beam bending due to the bend moving into the soil in response to the unbalanced thrust resultant?' absolutely nothing to do with the characterization of* *the pipe cross-sectional beam or axial bending behavior. In Chapter 9 of M9, one finds this footnote 'The factor ψ accounts for (1) tensile softening of concrete in tension, and (2) axial flexibility of the joint as joint rings and steel cylinder in tension slide relative to the core concrete over a bond development length when subjected to the design internal pressure $P = 1.25P_{weff}$. The value of ψ was determined conservatively from a comparison of the maximum effects of combined axial force and bending moment in bends calculated using the procedure described here with those of the axial force and bending moment determined from a finite-element procedure that accounts for tensile softening of concrete, axial flexibility of the joint, and side friction between the pipe and soil near the bend at the design internal pressure $P = 1.25P_{weff}$ per Zarghamee et al. (2004).' This footnote does not mention ACI 318 but provides a more elaborate description of this ψ accounting for multitude of secondary effects. How do you reconcile what is printed in this footnote and your answer above quoting ACI 318-11?"* The reply was, *"The use of a knock down factor of this magnitude is consistent with structural engineering practice when we are solving for strength under factored load when concrete loses stiffness due to tensile microcracking and possible cracking."*

Given that the above personal communication produced no worthy outcome, the author discontinued further efforts to get clarifications into what has been included in Chapter 9 of AWWA M9. The Chair of the M9 subcommittee and the standards engineer declared that the responder has made good faith efforts to answer the author's questions.

WEAKEST LINK IS GEOTECHNICAL REPRESENTATION

Backfill and in-situ soil types are classified into five different groups, referred to as soil types I through V and given in Table 9-1 of AWWA M9 (2008). Soil stiffness, k, accounts for the in-situ and the backfill soils combined. Values of k are based on finite-element soil-structure interaction analyses of pipe-soil systems. CANDE (1989) was used with the hyperbolic soil model. The above approach does not account for numerous factors that affect the stiffness constant needed for the soil-pipe interaction that takes place in the horizontal direction. Furthermore, the probability that a design engineer would be able to match both soil types listed for the backfill and the in-situ soil is a mere 4%. In all likelihood, the values given in Table 9-1 of AWWA M9 will be of no use to anyone in engineering practice, given the precision to which the numbers are chosen ignoring the enormous variability in soil conditions. Mere illusion of certainty is conveyed by choosing precise numbers like 425, 1100, 1900, 3400 and 7000 psi for k, while 20, 30, 34, and 48 degrees for the angle of internal friction. Even the unit weights have the same flavor – 110, 112, 114, 120 and 140 pcf. All these are used to represent the equivalent properties, k, ϕ and γ of two different soils for backfill-bedding and the native ground. Details of how the effective properties were determined, or under what conditions these hold true are not discussed in either Zarghamee et al.(2004), at their closure or in Chapter 9 of M9. Moreover, the evidence Zarghamee et al. (2005) relied on to defend the methodology and the k values from the literature cited in their closure simply do not exist. Examples are:

1) On page 1481 in the left column, Zarghamee et al. (2005) printed *"In 1961, Vesic analyzed an infinite horizontal beam-on-elastic foundation and related the soil stiffness to the modulus of sub-grade reaction. The results of that study are summarized in a book by Poulos and Davis (1990). Vesic recommends the values shown in Table 1 for an (sic) 2134 mm (84 in.) OD pipe with cover height from the midpoint of the pipe equal to 1.5 times the pipe diameter."* The References section of the closure appears on page 1482, where they cited the book by Poulos and Davis on page 174. There is, however, no Table 1 on p 174 or on any page in this book or in Vesic (1961) to defend the method and the k values Zarghamee et al.(2004, 2005) included that have been reproduced as Table 9-1 in AWWA M9 Chapter 9.

Table 1. Value of k in MPa (psi) Based on Vesic Formula

Type of sand/clay	Loose/stiff	Medium/very stiff	Dense/hard
Dry to moist sand	7.0 (1020)	21.1 (3,063)	56.3 (8,167)
Submerged sand	4.0 (583)	14.1 (2,042)	34.2 (4,959)
Clay	3.8-30.6 (555-4,444)	7.7-61.2 (1,110-8,880)	15.3-61.2 (2,200-8,880)

2) On page 1481 in the right column, Zarghamee et al.(2005) printed *"For seismic evaluation of buried pipe, O'Rourke and Liu(1999) show that the lateral soil stiffness depends on the stress level in the soil and recommend for low movement of the pipe that $k = 6.67p_u/y_u$. For a depth to diameter ratio of 1.5, the results of calculations show that the value of 10,500 psi for dense sand, 5,500 psi for medium sand, and 2,000 psi for loose sand."* The pretext in the source, however, reads, "For small ground movement such as the movement induced by wave propagation, El Hamdi and M. O'Rourke (1989) suggest the following soil spring constant: $k = 6.67p_u/y_u$. Zarghamee et al. (2005) left out the fact that this equation they quoted is only for extremely small movements during p and s wave velocity measurements in the field, and this level of movement has nothing to do with those in thrust restraint designs, which can be many orders of magnitude higher than those resulting from wave propagation. Furthermore, Zarghamee et al. (2005) admitted that the soil cover depth, diameter and stress level in the soil determine the magnitude of k, yet, this fact is not even mentioned in AWWA M9 Chapter 9. This is in complete contradiction to what Zarghamee et al. (2004) wrote, "Calculations performed for different soils, compaction levels, pipe diameters, and depths of cover, showing relative independence of k from pipe diameter, direction of pipe movement, and depth of cover for typical concrete pressure pipe diameters and typical installations."

3) In the right column of page 1481, Zarghamee et al. (2005) quoted O'Rourke's work to defend the choice of horizontal spring stiffness, k values using Vesic's vertical spring stiffness. In fact, O'Rourke accurately reports in his book that this equation applies for vertical spring stiffness. The lateral soil stiffness and the vertical soil stiffness are never the same.

Dana (2005) wrote in his discussion, "*In Zarghamee et al. (2004)'s eq. 3, the third term represents passive soil-pressure forces that act on the bend to resist thrust, T. The fourth term represents friction forces that act on the bend to resist thrust. Both are incorrect because they are not based on movement of the bend into the soil in the direction of the thrust force and because all bends are treated as having only a single mitered joint. The correct third term is = k δ l_k, where l_k is the effective length of the projection of the outer surface of the entire bend on a plane perpendicular to T. That projection is a rectangle of length $2l_b\cos(\Delta/2)$ and width Do, with half ellipses appended to its ends. The major axis of the ellipses is Do, and the minor axis is Do sin (Δ/2). The effective projected length, which is equal to the projected area divided by Do, is $l_k = 2\ l_b\cos(\Delta/2) + (\pi/4)\ D_o\sin(\Delta/2)$. The correct 4th term is $f_\mu l_\mu$, where l_μ is the sum of the centerline lengths of each of the mitered sections of the bends.*"Dana (2005) also wrote *"The authors depend upon axial friction forces on tied pipe from vertical loads We, Wp and Wf to resist thrust force, but they neglect friction forces produced by lateral soil pressure against the pipe. The total lateral forces producing friction along the springlines of pipe equal to 2Vo and because lateral pressure is greatest near the bend, it could be significant in reducing the amount of longitudinal reinforcement of the first new tied pipe as well as the number of tied pipe. It seems overly conservative to neglect the effect of friction from lateral soil pressure."*

Despite the guidance from Dana (2005), when the above method from Zarghamee et al. (2004) was added into AWWA M9 as Chapter 9 in the third edition, changes were made for the worse. The equation on equilibrium of forces left some terms out to become:

$$T = 2PA\sin\left(\frac{\Delta}{2}\right) = 2F_o\sin\left(\frac{\Delta}{2}\right) + 2V_o\cos\left(\frac{\Delta}{2}\right) + 2k\delta l_b\cos\left(\frac{\Delta}{2}\right) \quad (8)$$

l_b = centerline distance, in., is defined in AWWA M9 as from the point of intersection of the bend to the end of the mortar or concrete lining at the bell end of the first restrained joint. If l_b exceeds $2.5D_y\tan(\Delta/2)$ plus the distance from the end of the bend cylinder to the end of the mortar or concrete lining of the bell, use $l_b = 2.5D_y\tan(\Delta/2)$ and consider the bend laying length beyond l_b to be a restrained pipe attached to the bend by a welded joint.

IMPACT OF MISSING TERMS ON Fo

An in-depth study was undertaken by the author over several months with the enormous help of Dana to determine the impact of missing terms in the governing equation (9-6) in Chapter 9 of M9. The first step was to calculate the magnitude of f_μ for a broad range of conditions: depth of soil cover = 2 to 12 ft.; pipe size = 42 to 180 in.; μ = 0.3 to 0.5; R = 1.0 to 2.5Do; bend angle = 10 to 90 deg. Next, the term l_μ was calculated using l_b from C208 -07 & 12:

$l_\mu = 2\{l_b - R\tan(\Delta/2) + (n_s-1)\, R\tan(\Delta/[2(n_s-1)])\}$ AWWA C208-07 (9)

in which, l_b = L_1 which has two different forms:

For a two piece elbow, $L_1 = L + Z_2$ and

for a three, four or five piece elbow, $L_1 = L+Z_2+T-E$ (10)
l_b is $L= n_e Do\tan(\Delta/2)-Do\,(n_e-0.5)\tan(\Delta/(2(n_s-1)) +fDo$ AWWA C208-12 (11)
and l_b is used in $l_\mu= 2\{l_b-R\tan(\Delta/2) + (n_s-1)R\tan(\Delta/[2(n_s-1)])\}$ (12)

It appears that the impact of Dana (2005)'s fourth term ranges from 0.6 to 55.7% of design thrust, T using AWWA C208 (2007) and 0.9 to 59.4 % of T using C208 (2012), respectively. The next step was to estimate the effect of dropping the second part of the third term for a broad range of conditions: bend angle = 10 to 90 degrees; pipe size = 42 to 180 inch; soil = types I to V; δ = 0.003 to 2.75 in; P = 100 to 350 psi. The second part of the third term = $k\,\delta(\pi/4)Do\sin(\Delta/2)$; the third term's second part ranges from 0.2 to 7% of T. In summary, the missing second part of the third term and the entire missing fourth term add to 0.8 to 66.4% of T. A similar study was done to determine the impact of keeping the incorrect third and fourth terms of Zarghamee et al. (2004) versus leaving those two terms out and the results indicate that the third term accounts for a magnitude of 0.5 to 13% of T and that of the fourth term amounts to 0.5 to 29.7% of T. The contribution from the current third term in eq. 9-6 of AWWA M9 ranges from 0.7 to 40% of T. This means that the form of the equation (9-6) governing force equilibrium in the third edition of AWWA M9 Chapter 9 depends on the soil resistance term to be ranging from 0.7 to 40%, attributing the remaining 99.3 to 60% of T to the contribution from Fo and Vo; if the third edition did all of this the right way, then the soil resistance term would contribute an additional 0.8 to 66.4% of T, attributing as low as one half of the remaining 99.3 to 60% of T to the contribution from Fo and Vo. This implies, often the effect of leaving the second part of the third term and the entire fourth term amounts to the third edition of AWWA M9 Chapter 9 using the new thrust restraint method to over predict the magnitude of Fo and in turn requiring that the restraint length of the pipe recommended is as much as twice the length really needed in the field. This contradicts the claim Zarghamee et al. (2004) made in, "*a new design procedure is developed that provides an accurate method for the design of thrust restraint systems, and lends itself to engineering design calculations.*"

FINDINGS

1) The predominant claim the PCCP industry, Zarghamee et al. (2004), ACPPA and AWWA have about the new thrust restraint method in the third edition of AWWA M9 Chapter 9 is that this is the only method that checks for deformation compatibility. This claim has no merit when the force equilibrium, although far more essential, is based on an incorrect implementation of widely accepted geotechnical principles.

2) The magnitudes of the terms that are left out from the force equilibrium equation are much greater compared to the impact of insignificant shallow cover correction factor or the upper haunch soils. Preliminary runs using TRDP indicate that the answers are not the same as those from hand calculations, even on the design examples given in AWWA M9 Chapter 9; therefore, the author has chosen not to risk his professional license using TRDP when he does not know what is in the engine of

this software, given the problems he identified in the thrust restraint method formulation and implementation.

3) The dimensions of terms in the equations used in the three design examples given in Chapter 9 of M9 do not satisfy the Theorem on Dimensional Homogeneity.

4) The only experimental verification was on a 6 inch diameter 13 gauge steel pipe embedded in soils of 88 to 96% standard proctor density. Much of the verification of the validity of the results of FEA was accomplished by additional FEA, which raise doubts of the applicability of any of the results from Zarghamee et al. (2004) and AWWA M9 Chapter 9 to much larger and clearly diverse real world pipelines.

5) It is the author's professional opinion maintained all these years that the four failures of TRWD which Marshall (1998) reported due to thrust really have nothing to do with a lack of an adequate way to design the thrust restraint as it was offered in the AWWA M9 second edition. These failures were instead caused by the construction specifications allowing only flooding as the method of placing bedding and backfills in highly expansive clays.

6) The members of the AWWA concrete pressure pipe committee and ACPPA should be applauded for attempting to better understand the pipe-soil interaction, together with the stresses and strains, within the pipe material itself. The resulting method as presented in Chapter 9 of M9 third edition and the associated TRDP program, however, have fallen short of this goal. It has increased the level of confusion for the pipeline engineer wishing to perform thrust restraint design for concrete pipelines.

CONCLUSIONS

The author recommends the following changes in the third edition to encourage the wider use of AWWA M9 Chapter 9:

1) On page 132, Eq. 9-6 should use the original form proposed in Zarghamee et al. (2004) with the corrections mentioned in Dana (2005) applied.
2) On page 134, delete Eq. 9-8, given the shallow cover factor, β is insignificant.
3) On page 135, reformulate the use of moment of inertia reduction factor ψ to yield an outcome more realistic than the answers based on a constant value of 0.2.
4) On page 137, leave Table 9-1 out; although some other AWWA manuals include soil parameters, the author believes this table brings no value to practicing engineers other than unwarranted liability. Explain how k is defined, and the evidence that supports the values of k which exists in the published works.
5) On pages 141-145, provide all intermediate steps and bases for the assumptions and parameters of the calculation in Design Example 1, so that design engineers can build more confidence in this AWWA Manual of Practice.
6) On pages 146-150, rewrite the steps in Design Example 2 with updated equation for T, and correct estimates of the resistance coming from skin friction and passive resistance, leaving shallow cover correction factor.
7) On page 148, correct the units on both sides of the equations in the lower half of this page that do not presently satisfy the Theorem on Dimensional Homogeneity.

8) On page 151, remove the opening paragraph and encourage wider use of M9 Chapter 9 by showing all the calculations and not leaving essential items out; for example, withholding moment-rotation relationships undermines the trust the design engineer needs to develop while attempting to use AWWA M9 to design a project. Without such essential data disclosed, no user of this Manual of Practice M9 can follow the Design Example.
9) The author believes it is a poor practice to provide software that is essentially a black box. This interferes with the efforts of senior engineers mentoring younger engineers to think on their own. It is more conducive to the engineer's responsibility of charge to provide adequate documentation of the design method, as outlined in suggestion 8) above, so that a typical competent engineer can write his or her own engine within a spreadsheet program such as Microsoft Excel and use it to perform more efficient design calculations for thrust restraints.
10) The paramount objective of the writing team of a manual of practice is to serve the needs of the readers. A test group of engineers should be given drafts of the next edition of M9 Chapter 9 from time to time. Their feedback should steer the final version that gets printed and made available for dissemination.

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NOTICE

A discussion and closure of this paper are currently in development and will be published with the online version of this proceedings at http://dx.doi.org/10.1061/9780784479360

2014 Updates to ASTM C12

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Abstract

The ASTM standard C12 "Standard Practice for Installing Vitrified Clay Pipe Lines" gives recommended methods for installing vitrified clay pipe (VCP). C12 was significantly modified for the 2014 version (C12-14). Three of the most notable changes were:

- Use of uncompacted soil for the bedding
- Adoption of uniform soil groups
- Recommended gradation for the bedding soil

The current standard includes several bedding classes for proper support of the pipe with corresponding Load Factors. The Load Factors reflect the amount of support the bedding soil gives to the installation. While the basic trench configurations and Load Factors have not changed, the 2014 version incorporates various improvements including:

- Discontinuation of the concrete arch bedding class
- Adoption of *Uniform Soil Groups for Pipe Installation*
- Recommendation for methods of improving the foundation to provide proper support for the pipe when the foundation is deemed not suitable
- Updates to bedding standards, including:
  - Use of uncompacted soil for the bedding
  - Recommended gradation for bedding soils
  - Specifying the appropriate amounts of fractured faces for Class II soil particles
  - Limiting the maximum soil particle size for bedding and initial backfill materials to 1-inch and 1½-inch particles.
- Requiring shovel slicing of haunch area soil to occur before the bedding height is 0.25 of the outside diameter of the pipe.
- Replacing subjective language with more definitive language
- ASTM specialized standards relating specifically to Controlled Low Strength Materials (CLSM) bedding for VCP

In 1995, C12 became the first pipe installation standard to give CLSM bedding as a viable option. Requirements for flowability when placed, 28 day compressive strength, and set time prior to backfill load are now referenced to the ASTM CLSM standards. One advantage of using VCP is that it will not float during CLSM installation.

INTRODUCTION

One hundred years ago, in 1915, ASTM C12 was issued. Finally adopted in 1919, this standard has undergone countless revisions since. The standard was significantly modified in 2014 (C12-14).

The 2014 update incorporates the following improved installation requirements and techniques:

1- Discontinuation of the concrete arch bedding class.
2- Adoption of *Uniform Soil Groups for Pipe Installation.*
3- Recommendation for methods of improving the foundation to provide proper support for the pipe when the foundation is deemed not suitable.
4- Updates to bedding standards, including:
    a. Use of uncompacted soil for the portion of the bedding that the pipe is laid on.
    b. Recommended gradation for bedding soils.
    c. Specifying the appropriate amounts of fractured faces for Class II soil particles.
    d. Limiting the maximum soil particle size for bedding and initial backfill materials to 1-inch and 1½-inch particles, depending upon the bedding class.
5- Requiring shovel slicing of haunch area soil to occur before the bedding height is 0.25 of the outside diameter of the pipe.
6- Replacing subjective language.
7- Adopted ASTM standards relating specifically to CLSM.

Trench Cross Section

Figure 1 illustrates the terms that are used throughout this paper and in connection with the changes to the ASTM standard.

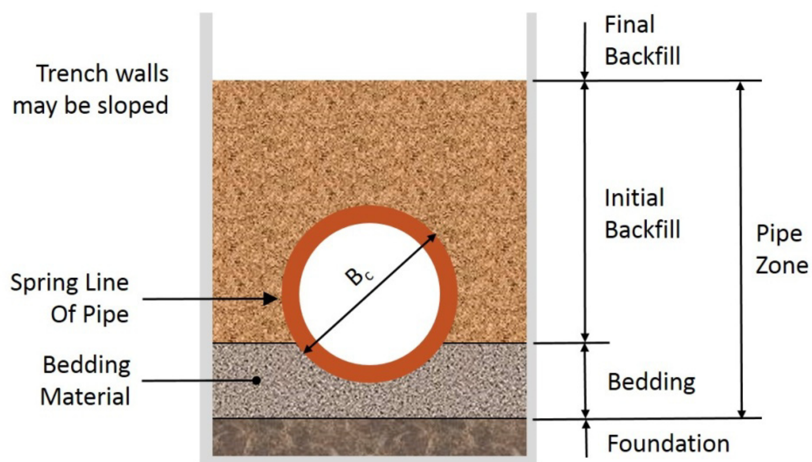

Figure 1: Trench Cross Section Terminology
(Class C shown)

Bedding Classes – Class D, C, B, and Crushed Stone Encasement

The previous standard included several bedding classes for proper support of the pipe with corresponding load factors. The currently recognized bedding classes are shown in Figure 2. The load factors reflect the amount of support the bedding soil gives to the installation (pipe bearing strength X load factor = field supporting strength). While the basic trench configurations and load factors have not changed, the references to type of soil used in the various areas have changed to use uniform soil group nomenclature. The soil groups are defined in Table 1 on page 4.

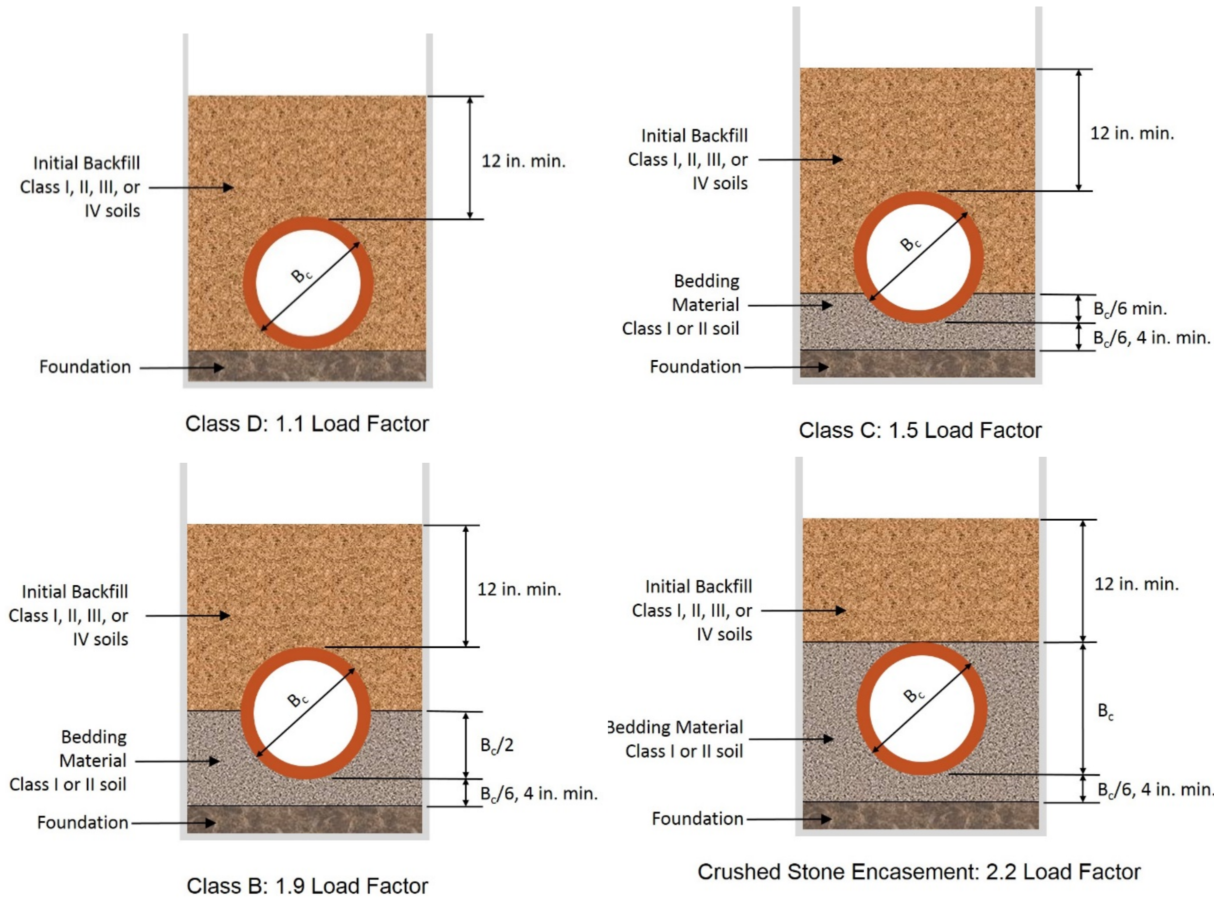

Figure 2: VCP Bedding Classes Class D through Crushed Stone Encasement

DISCONTINUED USE OF CLASS A – CONCRETE ARCH

Until the 2008 version of C12, Class A bedding was defined using two installation details, a concrete cradle and a concrete arch. The Class A concrete arch is no longer recommended and older specifications that include this option should be updated. The concrete arch was discontinued because any settlement of the bedding soil beneath the arch had the potential to increase the load on the pipe. Additionally, the settlement could result in a point load at the contact of the concrete arch on the top of the pipe. ASTM C12 continues to recognize two viable concrete bedding classes, concrete cradle and full concrete encasement.

UNIFORM SOIL GROUPS

The soil groups used in each bedding class are defined in Table 1. The soils are grouped according to strength required for pipe support (Howard 2009). This table has been incorporated into many ASTM standards and AWWA manuals and is now included in C12.

The additional soil requirements for VCP installations, such as gradation and particle angularity were added.

Uniform Soil Groups for Pipe Installation[1]

Soil Class	Definition	USCS Symbols
Class I[2]	**Crushed Rock** - 100% passing 1-1/2 in. (38 mm) sieve - $\leq$ 15% passing #4 sieve - $\leq$ 25% passing 3/8 in. (9.5 mm) sieve - $\leq$ 12% passing #200 sieve	
Class II[3]	**Clean, Coarse Grained Soils** - Any soil beginning with one of these symbols (can contain up to 12% fines) - Uniform fines sands (SP) with more than 50% passing a #100 sieve should be treated as Class III material	GW, GP, SW, SP
Class III	**Coarse Grained Soils With Fines** - Any soil beginning with one of these symbols **Sandy or Gravelly Fine Grained Soils** - Any soil beginning with one of these symbols, with $\geq$ 30% retained on #200 sieve	GM, GC, SM, SC ML, CL
Class IV	**Fine-Grained Soils** - Any soil beginning with one of these symbols, with < 30% retained on a #200 sieve	ML, CL
Class V[4]	**Fine-Grained Soils, Organic Soils** - High compressibility silts and clays, organic soil	MH, CH, OL, OH, Pt

[1] Soil Classification descriptions and symbols are in accordance with ASTM D2487 and ASTM D2488
[2] For Class I, all particle faces shall be fractured.
[3] Materials such as broken coral, shells, slag, and recycled concrete (with less than 12% passing a #200 sieve) should be treated as Class II soils.
[4] Class V soil is not suitable for use as a bedding or initial backfill material.

Table 1: Uniform Soil Groups for Pipe Installation

FOUNDATION

Trench load design for all pipe is based upon a firm and unyielding foundation. It is essential that the trench bottom remain stable during backfilling and under all subsequent trench operations. The foundation is critical to the performance of the entire pipe installation. The foundation must be firm and unyielding as it needs to support the bedding, pipe and backfill.

In cases where the trench bottom is soft and unsuitable to support the pipe, bedding and backfill; removal of material is necessary. Replacement can be accomplished with crushed rock or a woven geotextile fabric or both, to stabilize the foundation. Consult a Geotechnical engineer for other design methods to ensure the foundation supports the load.

The native material in the trench bottom must be capable of excavation to a uniform undisturbed flat bottom for a Class D installation. If the trench is over-excavated, the trench bottom should be brought back to grade with the required bedding material.

BEDDING

Uncompacted Bedding

A layer of uncompacted bedding beneath the pipe is now included for Class B and Crushed Stone Encasement. The weight of the pipe, the fluids in the pipe, and the backfill help the pipe settle into the uncompacted layer and create a small bedding angle for support. This settlement also mobilizes the strength of the haunch support. The use of uncompacted bedding is an acceptable and recognized technique as evidenced by its inclusion in many ASTM standards and AWWA manuals.

Accordingly the following statement was added to Section 9 "Bedding" under Construction Techniques:

> 9.2 The portion of the bedding directly beneath the pipe and above the foundation should not be compacted for Class B and Crushed Stone Encasement.

Gradation

The bedding soil shall be cohesionless soils, Class I or Class II. The gradation for Class I and Class II soil for Class C bedding shall have a maximum particle size of 1 in. (25 mm).

The gradation for Class I and Class II soil for Class B bedding, Crushed Stone Encasement, and CLSM bedding shall be as follows:

- 100% passing a 1 in. (25 mm) sieve
- 40-60% passing a 3/4 in. (19 mm) sieve
- 0-25% passing a 3/8 in. (9.5 mm) sieve

Particle Shape
Class II soils shall have a minimum of one fractured face. For Class B, Crushed Stone Encasement, and CLSM installations where high and/ or changing water tables are present, the composition of materials shall be as follows:

- 100% of the material shall have at least one fractured face,
- 85% of the material shall have at least two fractured faces,
- 65% of the material shall have at least three fractured faces.

The percent of fractured faces should be determined in accordance with ASTM D5821. Because the particles are 100% fractured, Class I material is considered to be more stable and provides better support than Class II material that may have some rounded edges.

HAUNCHING

Proper haunch support (Figure 3) is necessary for the achievement of the load factor and thus, the structural integrity of the pipe. Lack of proper haunch support is one of the most common causes of any pipeline failure.

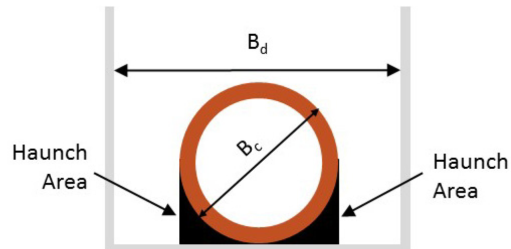

Figure 3: Pipe Haunch Areas

B_c = the outside diameter of the pipe.
B_d = the design trench width measured at the horizontal plane at the top of the pipe barrel.

Haunch support depends on three factors:

1. Proper compaction of the bedding materials in the pipe haunches
2. Mobilization of the bedding within the limits of the haunch area
3. Bell or coupling holes/ pipe barrel uniform support

Compaction Of Haunch Soil
Compaction of the soil in the haunch area significantly increases the support for the pipe. Gravels and crushed rock dumped into a trench beside the pipe result in minimum densities of the soil, which is about 80-85% of their maximum density. Compacting the soil to about 95% (ASTM D4253 *Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table*) can increase the stiffness (modulus) of the soil 300 to 600% (*Howard 2015*).

All bedding material shall be shovel-sliced so the material fills the haunch area and supports the pipe to the limits shown in the trench diagrams.

Shovel-slicing the bedding material in the haunch areas is critical. It takes little time, maintains grade, eliminates voids beneath the pipe and in the haunch areas, consolidates the bedding, and adds little or nothing to the cost of the installation. To be the most effective, and to meet the requirements of C12, shovel slicing should be done before the bedding is no higher than the quarter point of the pipe, as illustrated in Figure 4. Shovel-slicing the bedding material into the haunches of the pipe is essential if the total load factor of the bedding class is to be realized. Appropriate shovel slicing technique is demonstrated in Figure 5.

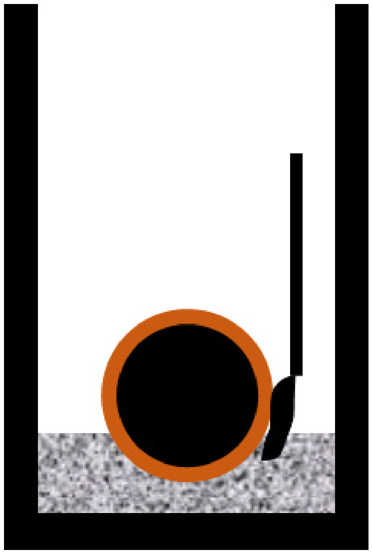

Figure 4: Initial haunching should be performed before the bedding is no higher than the quarter point of the pipe diameter.

Figure 5: Shovel-slicing the bedding material into the haunches of the pipe is essential.

Increased Haunch Support By Soil Mobilization

Haunch support for pipe can be effectively actuated by providing an uncompacted bedding for the pipe. The weight of the pipe, fluid in the pipe, and the backfill soil over the pipe help push the pipe into the uncompacted material creating a small cradle. Since uncompacted bedding under the pipe has a low stiffness, minor pipe settlement will mobilize the haunch soil support. Compacted haunch material is not as effective if the pipe is resting on compacted bedding above the foundation. The compacted soil simply acts as a filler. However, if the pipe is raised during compaction of the haunch soil, then the haunch support can be mobilized similar to uncompacted bedding. Uncompacted bedding material is specified directly beneath the pipe and above the foundation for both Class B and Crushed Stone Encasement classes in the current C12.

If uniformly graded gravel is loosely placed or dumped beside a pipe it will typically leave a void in the haunch area. This will reliably result in a decreased load factor no matter the bedding class. The gravel has an angle of repose, which is the angle of the slope of the material when dumped into a pile. Gravel with fractured faces will have a steeper angle than gravel with rounded edges. Figure 6 illustrates what happens when crushed rock with an angle of repose of 39 degrees is dumped beside a 36-in pipe with a 44-in outside diameter.

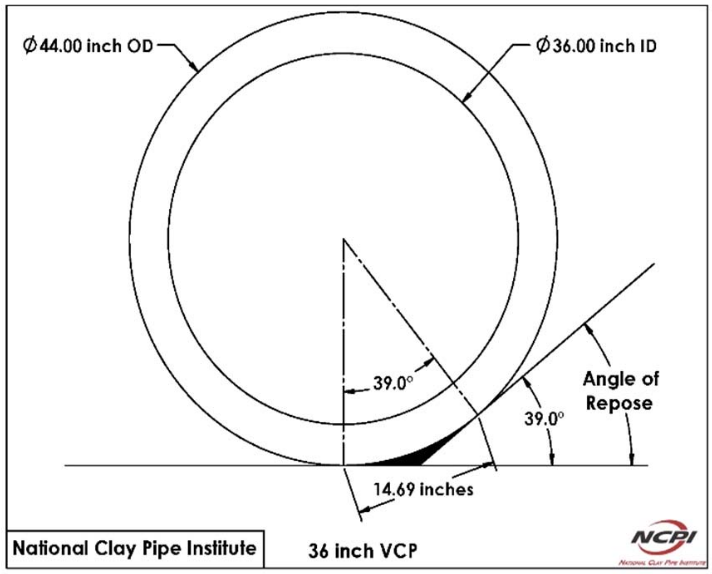

*Figure 6: Illustration of the 14.69" void space (**shaded black**) left in the haunches of a 44-in OD pipe when the bedding material angle of repose is 39 degrees and dumped alongside the pipe.*

Testing was conducted in 2013 to confirm the theory illustrated in Figure 6 (*Boschert and Howard, 2014*). Figure 7 is a photo from that research project. It clearly illustrates the reality of the haunch void. The photo was taken after the crushed rock had been dumped into a trench beside a 36-in pipe. Daylight can be seen on the other end of the pipe revealing a void running along the full length of the pipe in this lower haunch area. A video taken during this testing clearly demonstrates the mechanism of the formation of a void in this area. This video is available for viewing on the NCPI YouTube channel.

Good haunch support:

1. Significantly increases the load carrying capacity of buried pipe.
2. Requires compacting the soil in the haunch area, or using CLSM.
3. **Is not attained by dumping gravels and crushed rock beside the pipe.**
4. Can be attained by pipe settling into uncompacted bedding to mobilize the strength of the haunch soil.

Figure 7: In testing, daylight was visible on the other end of a 7.0' pipe section.

INITIAL BACKFILL

The initial backfill is the material placed from the top of the bedding to 12 inches above the top of the pipe. The soil can be Class I, II, III, or IV. Local materials may be used when the required load factor for the trench design can be achieved.

For initial backfill, bedding Class D installations require a maximum particle size of 1 inch while the other bedding classes require a maximum particle size of $1^1/_2$ inches. This reduced particle size in the Class D bedding detail is a result of the initial backfill beginning at the bottom of pipe and thus encompassing the pipe haunches. With Class D, many native materials taken from the trench will provide suitable support for clay pipe and may be the most cost efficient method of installation.

The initial backfill does not need to be compacted, especially over the top of the pipe. However, the final backfill may need to be compacted under roads, parking lots, etc. In that case, the initial backfill helps to serve as a padding over the top of the pipe.

CONTROLLED LOW STRENGTH MATERIAL (CLSM) BEDDING

In 1995 C12 became the first pipe installation standard to include CLSM bedding as a viable option. The accepted standard as practiced since that time is shown in Figure 8. Requirements for flowability when placed, 28-day compressive strength, and set time prior to backfill load are now specified in the standard.

For CLSM installations, the pipe shall be bedded on Class I or Class II soil. The bedding shall be placed on a firm and unyielding trench bottom and shall have a minimum thickness beneath the pipe of one-sixth of the outside pipe diameter, but not less than 4 in.

For pipe diameters 8 to 21 in, CLSM shall extend a minimum of 9 in on each side of the pipe barrel. For pipe diameters 24 in and larger, CLSM shall extend a minimum of 12 in on each side of the pipe barrel.

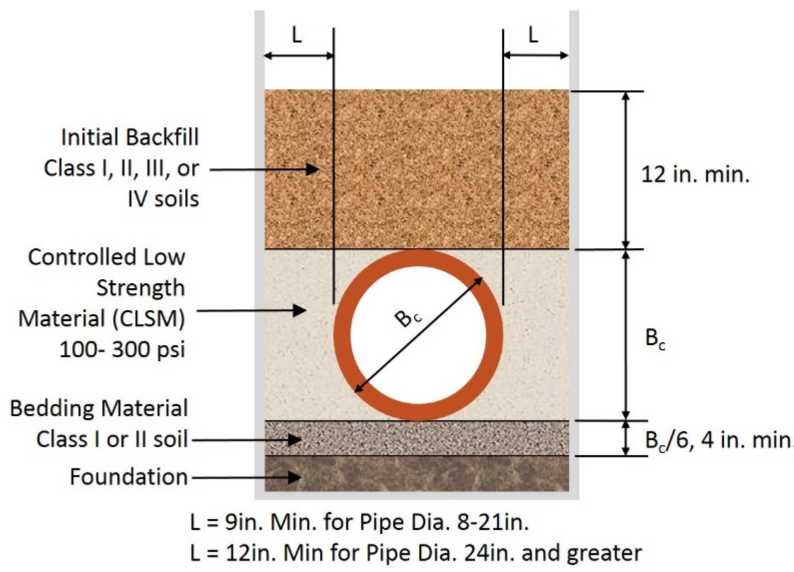

Figure 8: Controlled Low Strength Material (CLSM) Bedding: Load Factor 2.8

Figure 9: Measuring the spread diameter to determine flowability prior to placement.

Testing for flow consistency should be conducted in accordance with ASTM D6103. When placed, CLSM shall have a measured spread of 7 – 9 inches. A typical test result is shown in Figure 9.

The 28-day compressive strength shall be 100 to 300 psi as determined by Test Method ASTM D4832.

CLSM shall be directed to the top of the pipe to flow down equally on both sides to prevent misalignment. Place CLSM to the top of the pipe barrel.

Initial backfill shall only commence after a 500 psi minimum penetrometer reading is achieved as determined by Test Method C403/C403M. The penetrometer shall have a maximum load capability of 700 psi and have a 1.0 square inch by 1-inch long cylinder foot attached to a ¼-in diameter pin, as shown in Figure 10. The initial backfill shall be either Class I, II, III, or IV having a maximum particle size of 1½ in (38 mm).

Figure 10: A pocket penetrometer can be used to determine CLSM strength prior to backfill.

The fill can be made in a single pour to the top of the pipe or it can be made in two or more lifts if desired. No field installations using CLSM have resulted in flotation of clay pipe. However, buoyancy calculations done using the Archimedes' Principle (that a body wholly or partly immersed in a fluid is buoyed up with a force equal to the weight of the fluid displaced by the body) indicate that the pipe should have floated. Further research to date supports the theory that clay pipe does not float because CLSM acts as a Bingham fluid. A Bingham fluid, also known as a Bingham plastic, is a viscoplastic material that resists movement at low values of shear stress in the fluid. Buoyancy forces generate shear stress in the CLSM. If the stress applied by the buoyant force does not exceed the shear yield stress of the CLSM, the pipe will not float.

TABLE 2 ADDED TO C12

Table 2 was added to C12-14 to summarize the soil requirements for the various bedding classes and serve as a quick reference. It summarizes the type of allowable soil, maximum particle size, and gradation requirements for the bedding and initial backfill materials as discussed in detail in the Bedding section of this paper.

Allowable Bedding Material and Initial Backfill per Bedding Class					
Bedding Class	**Allowable Bedding Material**			**Allowable Initial Backfill**	
	Class (Table 1)	**Gradation**	**Particle Size**	**Class** (Table 1)	**Particle Size**
Class D	N/A	N/A	N/A	I, II, III or IV	1” (25mm)
Class C	I or II		1” (25 mm)	I, II, III or IV	1½” (38 mm)
Class B	I or II	- 100% passing a 1” (25 mm) sieve - 40 - 60% passing a ¾” (19 mm) sieve - 0 - 25% passing a 3/8” (9.5 mm) sieve	1” (25 mm)	I, II, III or IV	1½” (38 mm)
Crushed Stone Encasement	I or II		1” (25 mm)	I, II, III or IV	1½” (38 mm)
CLSM	I or II		1” (25 mm)	I, II, III or IV	1½” (38 mm)
Cradle	N/A	N/A	N/A	I, II, III or IV	

Table 2: Allowable Bedding Material and Initial Backfill per Bedding Class

ONGOING AND FUTURE RESEARCH

The ‘Importance of Haunching’ project of 2013 was conducted to determine the difference in haunch soil relative density when dumped alongside the pipe as compared to sliced directly into pipe haunch using a shovel. In these experiments, shovel slicing in the haunch increased the support for the pipe about tenfold as compared to dumped without slicing. A continuation of this project in under way using the saturation and vibration method for haunch compaction of both Class I and Class II soils. Additionally, the use of native soils as the aggregate/soil in CLSM mixtures will be examined.

SUMMARY

The latest version of the “*Standard Practice for Installing Vitrified Clay Pipe Lines*”, C12-14, reflects current technology and language to aid the designer, specification writer, contractor and inspector in delivering a successful installation. The changes include:

- Discontinuation of the concrete arch bedding class
- Adoption of *Uniform Soil Groups for Pipe Installation*
- Recommendation for methods of improving the foundation to provide proper support for the pipe when the foundation is deemed not suitable
- Updates to bedding standards, including:
    - Use of uncompacted soil for the bedding

- Recommended gradation for bedding soils
- Specifying the appropriate amounts of fractured faces for Class II soil particles
- Limiting the maximum soil particle size for bedding and initial backfill materials to 1-inch and 1½-inch particles.

- Requiring shovel slicing of haunch area soil to occur before the bedding height is 0.25 of the outside diameter of the pipe.
- Replacing subjective language with more definitive language
- ASTM specialized standards relating specifically to CLSM bedding for VCP

A table was added that summarizes the type of allowable soil, maximum particle size, and gradation requirements for the bedding and initial backfill materials.

For more information on any of these standards and recommended installation practices, see the latest edition of the National Clay Pipe Institute's *Vitrified Clay Pipe Engineering Manual* (2015), currently available on the website at ncpi.org.

REFERENCES

ASTM C 12 *Standard Practice for Installing Vitrified Clay Pipe Lines*

ASTM C 403 *Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance*

ASTM D 4253 *Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table*

ASTM D 4832 *Test Method for Preparation and Testing of Controlled Low Strength Material (CLSM) Test Cylinders*

ASTM D 5821 *Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate*

ASTM D 6103 *Standard Test Method for Flow Consistency of Controlled Low Strength Material (CLSM)*

ASTM D 7382 *Test Method for Dry Density of Granular Soils Using a Vibratory Hammer*

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Numerical Analysis of Pipe-in-Pipe Filled with Various Materials

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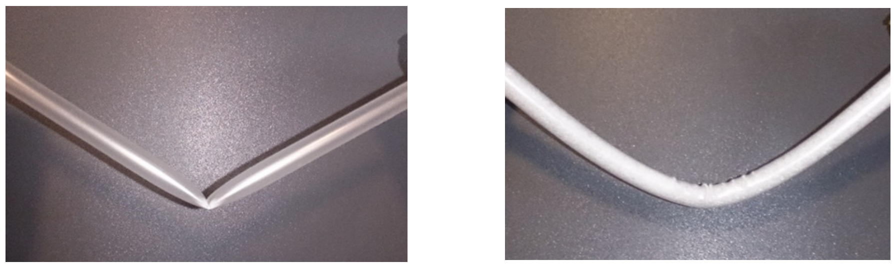
Abstract

The pipe-in-pipe proposed in this paper is an innovative structural material with more flexible and ductile bending performance than that shown by usual steel pipes. A pipe-in-pipe structure comprises double thin-walled pipes with granular material such as sand filled between the outer and inner pipes. The filling material prevents local failure known as the Brazier effect by transmitting the interactive stress between the pipes. As a result, smooth and flexible bending deformation is realized even under a large bending moment. In addition, when the sand filled in the pipe is frozen, the flexural rigidity of the pipe increases, but the pipe maintains its high ductility. To adopt such a pipe-in-pipe as a practical pipeline material, a numerical method must be established to evaluate the elastoplastic bending behavior. This paper proposes numerical methods for the bending behaviors of pipe-in-pipes and examines the difference in bending behaviors due to the filling materials using the proposed models.

INTRODUCTION

One of the typical failures of a pipe subjected to a bending moment is local buckling known as the Brazier effect, in which the sectional shape of the pipe partially acquires an oval shape [Brazier, 1926]. Then, the pipe loses its resistance against bending even if the rest of the pipe (i.e., except for the buckled portion) retains its original flexural rigidity. Picture 1 shows the bending deformations of a straw to demonstrate the effect of the filling material. The pipe-in-pipe design has already been applied mainly to riser pipes of offshore structures because a large flexural rigidity can be exerted by filling the core with high-strength material [Makino et al., 1993], [Ishii et al., 1996], [Li, 1997]. The elastic buckling failure of these pipe-in-pipes was theoretically examined by Sato to ensure their structural stability under hydrostatic high pressure [Sato et al., 2007].

The flexible and ductile pipe-in-pipe proposed in this paper consists of double tubes, and the gap between the two tubes is filled with various materials such as granular sand, mortar, and frozen sand, as shown in Figure 1. The authors previously confirmed through bending experiments that a pipe-in-pipe filled with granular sand can bend smoothly and uniformly without local buckling under a large curvature because the filled material constrains the cross-sectional deformation of the tubes by transmitting stress [Kanie et al., 2006]. In addition, when the sand filled in the pipe is frozen, the flexural rigidity of the pipe increases, although the pipe maintains its high ductility. To adopt such a pipe-in-pipe design for a practical pipeline, a numerical method must be developed to evaluate the bending behavior. This paper proposes numerical methods for the bending behaviors of pipe-in-pipes and examines the difference in the bending behaviors due to the filling materials by using the proposed model.



(a) Hollow straw without filling material (b) Straw filled with sugar

Picture 1. Effect of filling material on the bending behavior of straw.

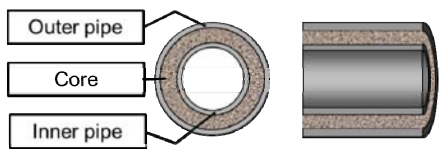


Figure 1. Pipe-in-pipe.

BENDING EXPERIMENTS

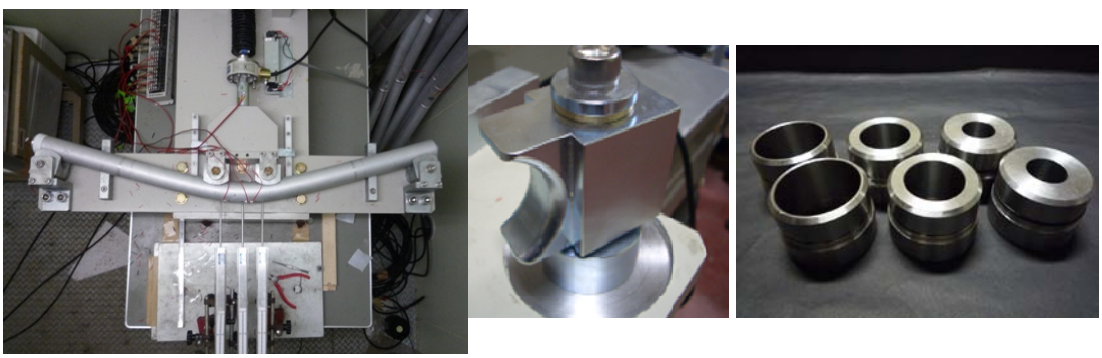
Testing apparatus. To confirm the flexibility and ductility of the pipe-in-pipe, we performed indoor experiments to examine the relationship between the bending moment and bending curvature of the pipe. The testing apparatus is shown in Picture 2. The outer pipe of the specimen is supported by bending free supports at both ends, and two-point loading is applied at the mid-span. The bending moment, which is constant between the two loading points, is calculated based on the pushing force of the actuator, the magnitude of which is measured by a load-cell. The bending deformation is monitored by three displacement gauges set between two loading points, and the bending curvature can be computed using these gauges. To observe the strain of the pipe during experiments, the pipe was equipped with strain gauges, and the results were used for verification of the bending curvature. The end caps shown in Picture 2 were used to keep the outer and inner pipes at the right position for the pipe-in-pipes.

Test specimens. First, three different types of single-walled pipes were examined: SGL-HL, a single-walled hollow pipe without filling; SGL-MT, a single-walled pipe filled with mortar; and SGL-SD, a single-walled pipe filled with sand. In these cases, we did not apply an inner pipe; however, the effect of the filling material on the bending behavior was investigated. Each specimen with a length of 1 meter as made of aluminum alloy, and the outer diameter was 50 mm with a thickness of 1 mm.

Second, we prepared three different pipe-in-pipes filled with granular sand. The dimensions of the outer pipes were identical to those of the single walled pipes used for the SGL specimens. However, the diameters of the inner pipes were different. For

DBL-SD-ϕ20, DBL-SD-ϕ30, and DBL-SD-ϕ40, the diameter of the inner pipe was 20, 30, and 40 mm, respectively. The longitudinal axes of the outer and inner pipes should be identical and located at the center of the cross section before the loading. We provided end caps (Picture 2) to hold the outer and the inner pipes at the right positions. These end caps were used at both ends of the pipe-in-pipes. When the core was filled with sand, its density was carefully controlled. Toyoura standard sand, which is regulated by an old Japan Industrial Standard (JIS Z 8801), was used as the filling material. We compacted the sand by knocking the outer pipe with a soft hammer by gradually pouring sand into the core, and the sand densities were maintained between 1.57 [g/cm^3] and 1.60 [g/cm^3] as its dense condition.

The test specimens of the pipe-in-pipes filled with frozen sand were named as follows: DBL-FS-ϕ20, DBL-FS-ϕ30, and DBL-FS-ϕ40 indicate that the diameter of the inner pipe was 20, 30, and 40 mm, respectively. After sand filling, we poured distilled water into the core to satisfy the saturation intensity of 95%. Anti-freezing liquid at a temperature of -10 °C was poured through the inner pipe, and the sand was frozen from the inside for 72 hr. Before the experiment, the specimens were stored in a refrigerator at a constant temperature of -10 °C for more than three days to attain a constant temperature distribution. The loading rates of the DBL-FS series were set at 0.23 mm/min, which is the slowest rate of the actuator. The specifications of the specimens are tabulated in Table.1.



(a) Loading apparatus (b) Support (c) End cap

Picture 2. Bending experiments.

Table.1 Specifications of specimens.

Case name	Diameter of inner pipe (mm)	Filling material	Loading rate (mm/min.)	Total # of specimens
SGL-HL	NA	None (Hollow)	1.0	2
SGL-MT	NA	Mortar	1.0	2
SGL-SD	NA	Sand	1.0	2
DBL-SD-ϕ20	20	Sand	1,0	2
DBL-SD-ϕ30	30	Sand	1.0	2
DBL-SD-ϕ40	40	Sand	1.0	2
DBL-FS-ϕ20	20	Frozen Sand	0.23	1
DBL-FS-ϕ30	30	Frozen Sand	0.23	1
DBL-FS-ϕ40	40	Frozen Sand	0.23	1

All of the pipes were made of aluminum alloy with a length of 1 m.

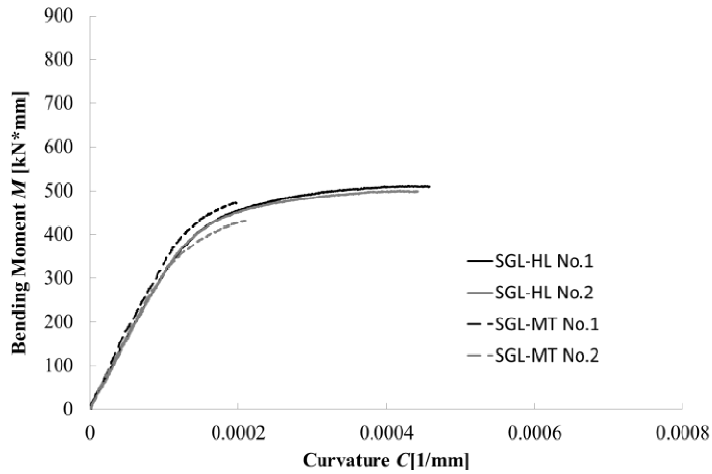
EXPERIMENTAL RESULTS

Single-walled pipe. Figure 2 (a) shows the relations between the bending moment and bending curvature for SGL-HL and SGL-MT. The flexural rigidity of SGL-MT filled with mortar was slightly improved compared with that of SGL-HL; however, it failed at very low curvature. In contrast, SGL-HL, a hollow pipe, could bend to much larger curvature without failure. The mechanism of this behavior can be explained as follows: (1) the mortar contributes to the initial increase in the flexural rigidity because it can resist against both compressive and tensile stresses in the longitudinal direction due to bending; (2) when the tensile stress reaches the critical strength, the flexural rigidity suddenly deteriorates locally and partially; and then, (3) the bending curvature is drastically accumulated at the local portion of the pipe where the crack in the mortar occurred even though the remainder of the pipe retains its sound condition. In other words, the critical bending moment for failure depends on the tensile strength of the filling material when a continuous material such as mortar is used as the core material.

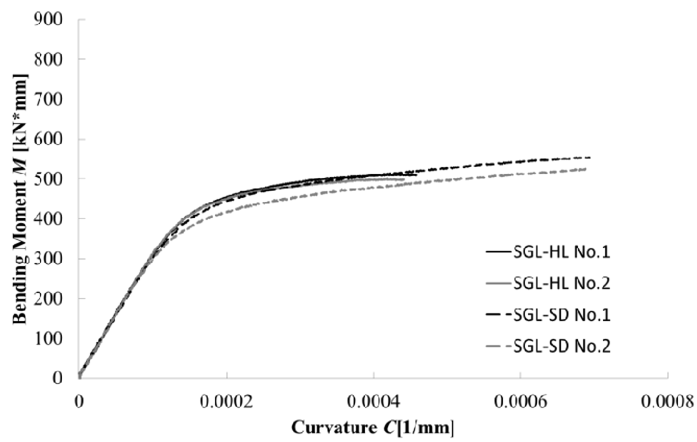
The role of sand as a filling material is interesting. The effect of sand on the ductility of the pipe is surveyed by comparing SGL-HL and SGL-SD. Figure 2 (b) shows the relation between the bending moment and bending curvature. During the elastic deformation, no difference can be observed between these parameters. However, the critical bending curvatures of SGL-SD filled with sand are much larger than those of SGL-HL. The sand as a filling material rarely increases the flexural rigidity because sand is not a continuous body and has no tensile strength. The compressive stress acting in sand in the longitudinal direction is negligibly small for the flexural rigidity of the pipe. The authors thought that the sand filled in the pipe prevented the deformation of the pipe within the cross section, similar to the Brazier effect. Consequently, the pipe filled with sand, SGL-SD, bends smoothly without local failure, and the critical bending curvature becomes much larger than that of SGL-HL.

Pipe-in-pipe. Similar to the experiments for single-walled pipes, we performed experiments for pipe-in-pipes filled with sand. Figure 3 (a) shows the relation between the bending moment and bending curvature compared with those of SGL-SD. For the pipe-in-pipes, the flexural rigidity is determined by the combination of the outer and inner pipes. As shown in Figure 3 (a), the flexural rigidity in the elastic range of DBL-SD-ϕ40 and the bearing bending moment in the plastic range are the largest. Similarly, the bearing bending moment increased with increasing diameter of the inner pipe.

The critical bending curvature of DBL-SD-ϕ40 was 0.0007 [1/mm], which is almost the same as that of SGL-SD, a single-walled pipe filled with sand. Even though the critical bending curvature of DBL-SD-ϕ30 was a little smaller than that of DBL-SD-ϕ40 and SGL-SD, we did not find any symptom of the Brazier effect in the failure of DBL-SD-ϕ30. We are now evaluating the reason for this behavior, and some additional experiments for DBL-SD-ϕ30 may be necessary. However, it is confirmed that the pipe-in-pipes filled with sand bend smoothly with a larger critical curvatures than those of a single-walled pipe.



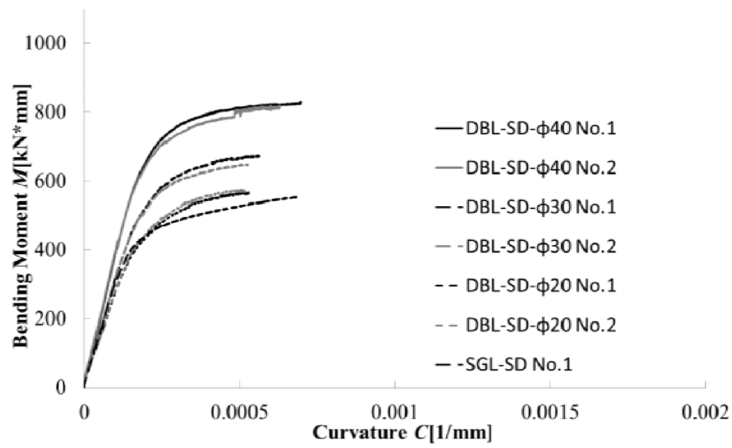
(a) SGL-HL and SGL-MT



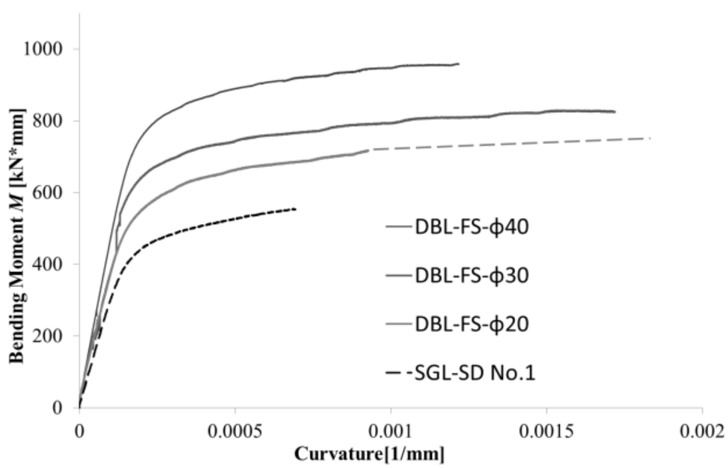
(b) SGL-HL and SGL-SD

Figure 2. Relation between bending moment and curvature of SGL.

The relations between the bending moment and curvature when the filling sand was frozen are shown in Figure 3 (b) with the experimental results of SGL-SD for comparison. By freezing the filling material, the bearing bending moments as well as the critical bending curvatures increased in all cases. DBL-FS-ϕ20 bent so much that the displacement at loading points exceeded the loading stroke of the actuator, and the specimen was not broken. We measured the curvature directly after the loading and plotted the maximum bending moment in Figure 3 (b) with the broken line for DBL-FS-ϕ20.



(a) DBL-SD



(b) DBL-FS

Figure 3. Relation between bending moment and curvature of DBL.

FINITE ELEMENT METHOD WITH FIBER MODEL

For practical application of pipe-in-pipe filled with various materials, the establishment of a numerical model is essential to appropriately and consistently evaluate the bending behavior from elastic to plastic ranges. The authors propose a numerical model based on the experimental results.

FEM with fiber model. It is preferable that the numerical model consistently covers the elastic and plastic ranges of deformation. In the evaluation of bending deformation of the pipe-in-pipe, the elastoplastic behavior of the material in the longitudinal direction should be consistently estimated. Therefore, the Finite Element Method (FEM), with a fiber model was introduced. The cross-section was divided into small sections, and each of those sections was modeled as a fiber element in the longitudinal direction.

As a beam deformation theorem, the Timoshenko beam, which considers both bending and shear deformations, was adopted [Timoshenko et al., 1959], [Timoshenko et al., 1970]. First, we introduced displacements of u, v, and w as functions of the longitudinal location in the x, y and z directions, respectively. Similarly, θ_x, θ_y, and θ_z are the rotation angles of displacements around the axes indicated by their subscripts. The displacements at the coordinates of x, y and z can be evaluated using Equation (1) to (3). Here, U in Equation (1) is the displacement in the x direction, V in Equation (2) is the displacement in the y direction, and W in Equation (3) is the displacement in the z direction. The variable ω is a warping function. For the interpolation in the longitudinal direction, the linear shape function is adopted to prevent shear locking [Nonaka et al., 2010].

$$U(x,y,z) = u(x) - y\theta_z(x) + z\theta_y(x) + \omega(y,z)\theta'_x(x) \tag{1}$$

$$V(x,y,z) = v(x) - z\theta_x(x) \tag{2}$$

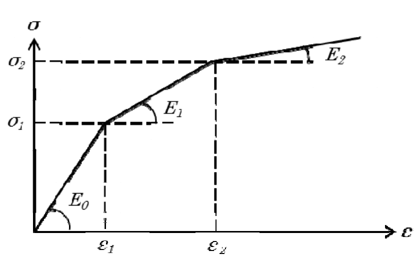
$$W(x,y,z) = w(x) + y\theta_x(x) \tag{3}$$

This model evaluates the bending behavior consistently from the elastic to plastic ranges in the material property even if the bending deformation becomes large. Finally, the element stiffness matrix $[k]$ is described by Equation (4). Here, $[B_l]$ is the infinitesimal displacement-strain matrix, and $[D]$ includes the stress and strain relations of the materials.

$$[k] = \iiint [B_l]^T [D][B_l] dxdydz \tag{4}$$

To apply the FEM model, the cross section was divided into 24 elements in the circumferential direction, and the longitudinal length was discretized into 50 elements with a longitudinal interval of 20 mm.

Material property of pipe. All the pipes used in our experiments were made of aluminum alloy. First, we examined its material property. Based on the experimental results of SGL-HL, the stress and strain relation of the material was discussed. We assumed a tri-linear relation for the stress and strain relations and attempted to find an appropriate model by data fitting with the experimental results. Figure 4 shows the stress and strain relation of the aluminum alloy used in the experiments after the data fitting. If those values are correctly fixed, the bending behavior of a single-walled pipe can be properly estimated using the numerical model described in the next section.



$E_0 = 60.0GPa$ — E_0: Preliminary Young's modulus
$E_1 = 17.1GPa$ — E_1: Secondary Young's modulus
$E_2 = 1.50GPa$ — E_2: Tertiary Young's modulus
$\varepsilon_1 = 0.00289$ — ε_1: Preliminary yielding strain
$\varepsilon_2 = 0.00486$, — ε_2: Secondary yielding strain
$\sigma_1 = 0.164GPa$ — σ_1: Preliminary yielding stress
$\sigma_2 = 0.215GPa$ — σ_2: Secondary yielding stress

Figure 4. Tri-linear stress-strain relation.

Verification of material property and fiber modeling. If the material property and FEM with fiber modeling are properly assumed, the bending behavior of a single-walled pipe filled with sand, SGL-SD, can be numerically estimated to follow the experimental result. A comparison of the numerical estimation with the experimental results is presented in Figure 5. The results are almost identical to each other and it is verified that the material property is accurately assumed and that the numerical model can appropriately simulate the bending deformation.

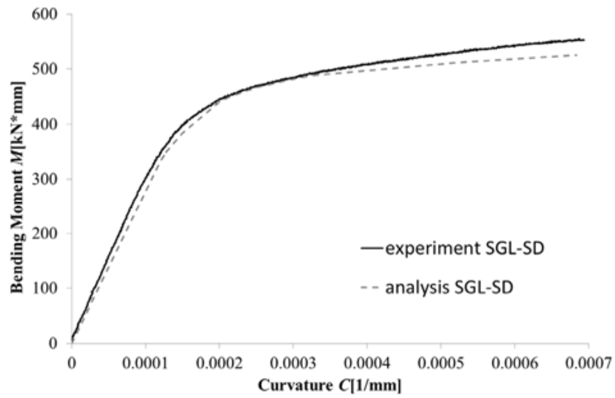


Figure 5. Relations between bending moment and curvature of SGL-SD.

NUMERICAL MODEL FOR GRANULAR MATERIAL

The interaction due to the filling material between the outer and the inner pipes should be appropriately expressed in the numerical model to satisfy the hypothesis that the curvature of the inner pipe is not always the same as that of the outer pipe. In our experiments, the bending moment is given on the outer pipe, and the inner pipe bends as a reaction through the filling material. Accordingly, we inserted a Winkler spring between the outer and inner pipes to represent the interaction due to the sand within the cross section. However, it rarely works in the longitudinal direction and hardly contributes to the increase in the flexural rigidity. Figure.6 shows the Winkler spring model.

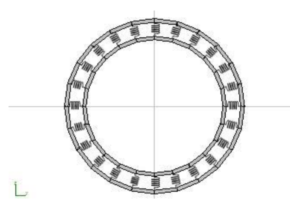
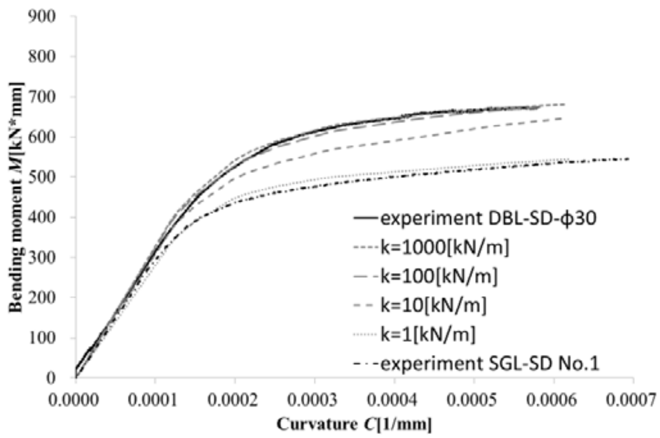


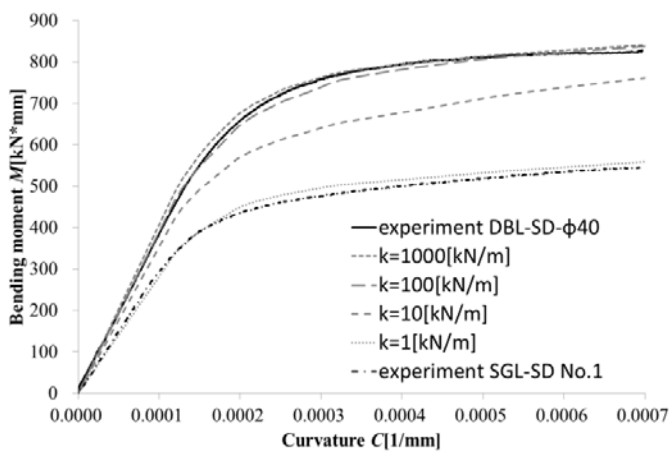
Figure 6. Winkler spring model for granular material.

The most important aspect of the numerical simulation is setting the spring coefficient of the Winkler spring. If the value is assumed to be zero, no interaction occurs between the pipes, and the inner pipe never bends even after the outer pipe yields. In this case, the flexural rigidity as a pipe-in-pipe is calculated in the same manner as that of the SGL-SD by neglecting the existence of the inner pipe. In contrast, if the value of the spring coefficient is assumed to be infinity, both the outer and inner pipes bend together while sharing the same longitudinal axis and maintaining their relative distance constant. The estimated result for the flexural rigidity becomes identical to that of the superposition model. We assumed various values for the spring coefficient such as k=1, 10, 100, and 1000 [kN/m].

Figure 7 shows the relations between the bending moment and bending curvature calculated using the numerical model with various spring coefficients, and the results are compared with the experimental results. When the lowest value of k=1 [kN/m] is used for the spring coefficient, the estimated results are very close to the experimental results of SGL-SD. Upon increasing the spring coefficient, the flexural rigidity is improved, and the simulated results for k = 1000 [kN/m] are almost the same as the results obtained using the superposition model.



(a) DBL-SD-ϕ30



(b) DBL-SD-ϕ40

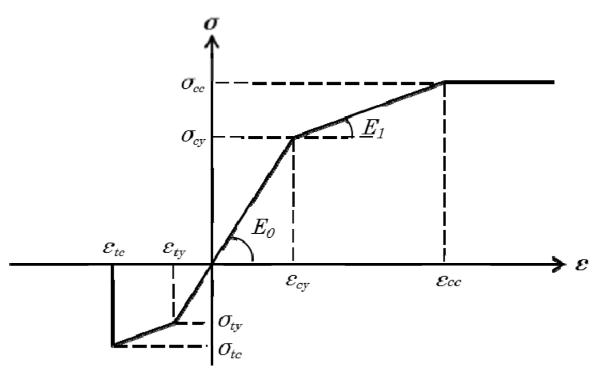
Figure 7. Relations between bending moment and curvature of DBL-SD.

Based on these figures, a spring coefficient of k = 100 [kN/m] to 1,000 [kN/m] is recommended to simulate the interactive action due to the filling material. Because a Winker spring is a discretely distributed spring, we calculated the reaction coefficient of the foundation that is equivalent to the Winkler spring by considering its distribution density. In our simulation model, the value of k = 100 [kN/m] for the Winkler spring is equivalent to k_f = 10 [kgf/cm^3] as the reaction coefficient of the foundation. Based on the density of the filling material of 1.57 [g/cm^3] to 1.60 [g/cm^3], the estimated value of the reaction coefficient of k_f = 10 [kgf/cm^3] is usually adopted for sand foundation compacted with a very high density. The authors are convinced that the numerical evaluation model for a pipe-in-pipe with a Winkler spring successfully simulates the bending behavior consistently from the elastic to plastic ranges in strain.

NUMERICAL MODEL FOR FROZEN SAND

When frozen sand is used as the filling material, we adopted a fiber model similarly to the pipe material. However, frozen sand has a larger compressive strength rather than tensile strength. Ueda et al. reported the moduli of deformation of frozen soil observed during indoor experiments [Ueda et al., 2007]. When dense Toyoura sand, which is widely used as standard sand in Japan, is frozen, the stress-strain relationship can be modeled by a bilinear approximation on the compressive side unless the strain is smaller than the critical compressive strain ε_{cc}. Because the loading rate of our experiments is slow enough to expect ductility in compression, we assume that the stress remains constant after the critical compressive strain ε_{cc}.

For the tensile side, Akagawa et al. conducted expansion experiments of frozen sand and reported that the tensile strength of frozen sand is almost 1/10 of the compressive strength [Akagawa et al., 2009]. Then, the authors assumed that the tensile yielding stress σ_{ty} and tensile critical stress σ_{tc} were proportional to those on the compression side with a coefficient of 1/10 while maintaining the same gradients of E_0 and E_1. However, when the tensile strain exceeds the tensile critical strain ε_{cc}, no tensile stress is exerted. The stress-strain relationship of frozen sand shown in Figure 8 is finally assumed.



$$E_0 = 4.06GPa$$
$$E_1 = 1.2GPa$$
$$\varepsilon_{yc} = 0.002$$
$$\varepsilon_{tc} = 0.0057$$
$$\varepsilon_{yt} = -0.0002$$
$$\varepsilon_{tt} = -0.00057$$

Figure 8. Stress-strain relationship for frozen sand.

The calculation results are illustrated in Figure 9 together with the experimental results. The calculation results coincide well with the experimental results; however, the estimations for DBL-FS-ϕ20 in the plastic range are slightly underestimated. The numerical method proposed by the authors can evaluate the elastoplastic bending behavior consistently with enough accuracy even if the filling material is frozen.

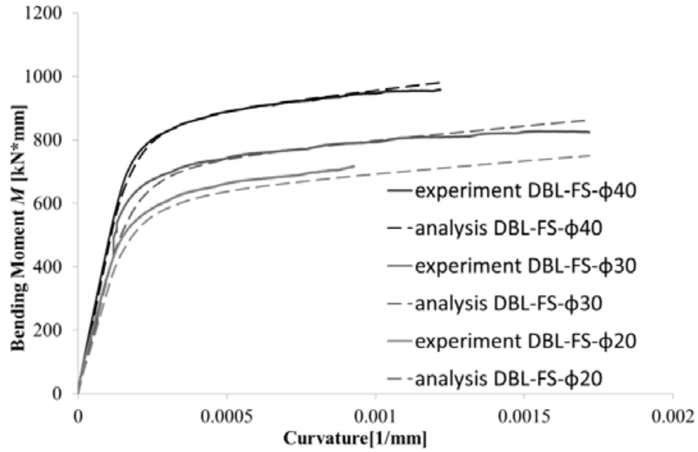


Figure 9. Relations between bending moment and curvature of DBL-FS.

CONCLUSION

The pipe-in-pipes proposed in this paper can bend smoothly to a large bending curvature without any local failure. When a granular material is filled in the pipe-in-pipe, the constraining effect due to the filling material can be estimated by a Winkler spring, whose coefficient is equivalent to the coefficient of reaction for sand foundation. If the sand as a core material is frozen, applying the fiber model approximation for the frozen sand is recommended. Throughout this study, the numerical models evaluated the elastoplastic behavior of bending, and the simulation results were verified as the experimental results with high flexibility and ductility.

The pipe-in-pipe filled with a granular material has already been registered as a domestic patent in Japan in 2013 by the authors. It can be applied as a structural member where large residual displacement occurs due to earthquakes by faults and/or due to frost heave in discontinuous permafrost [Williams, 1989]. The structural dimensions of indoor experiments that we performed were rather small when its practical applicability to a real field is discussed. The size effect of the model may be considered in the evaluation of interaction due to the filling material. The authors are planning further studies to enlarge the practical applicability using real-scale models.

ACKNOWLEDGEMENT

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Design and Construction Case History—South Catamount Transfer Pipeline Float-Sink

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Abstract

The Blue River Pipeline conveys water from Montgomery Reservoir near the Continental Divide over 70 miles by gravity to reservoirs on the north slope of Pikes Peak. The original transfer system from the Blue River Pipeline, constructed in the early 1950's, consisted of parallel 14-in. and 16-in. steel pipes to South Catamount Reservoir, crossing North Catamount Creek in the process. In about 1959, North Catamount Dam and Reservoir were constructed, inundating the North Catamount Creek crossing of the transfer pipeline with up to 75 feet of stored water. Over the ensuing decades, the pipeline deteriorated, resulting in leakage to the point that water could no longer be effectively transferred to South Catamount Reservoir, and replacement or rehabilitation of the pipeline was required. Due to seasonal and other constraints, design-build procurement was selected. The recommended pipeline alternative was 36-in (DIPS) DR11 HDPE, placed across North Catamount Reservoir using the float-sink method. In this method, the HDPE pipe is floated into position with the ends capped and with concrete ballast spaced along the length of the pipe to achieve about 50% buoyancy. Once the pipe is floated into position, it is filled with water, causing it to sink to its final position at the bottom of the reservoir. The project construction was facilitated by historically low reservoir levels as a result of drought, which shortened the length and depth of float-sink required. A long, relatively straight access road to the reservoir crossing allowed the entire length of HDPE to be fused prior to launching. Ballast consisted of single-piece concrete blocks placed beneath the pipe, affixed with stainless steel straps, rather than the more conventional two-piece concrete ballast. Sinking was completed in mid-November 2013, just prior to ice formation on North Catamount Reservoir. Total length of HDPE placed beneath the reservoir was 2,450 feet.

BACKGROUND

The South Catamount Transfer Pipeline (Transfer Pipe) is part of a system to transfer water from the Blue River drainage in Summit County on the west side of the Continental Divide to terminal storage reservoirs on the North Slope of Pikes Peak. South Catamount Reservoir was constructed by the City of Colorado Springs in about 1935, along with the companion Crystal Creek Reservoir, to collect and store local yield from the north slope of Pikes Peak. In about 1955, the Blue River Pipeline was constructed to convey the Blue River water. The Blue River Pipeline is approximately 70 miles long and delivers water by gravity at minimal operating cost. At the downstream termination of the Blue River Pipeline, an atmospheric vault was constructed, and water was conveyed from this vault to South Catamount Reservoir in two steel pipes, 14-inch and 16-inch, in parallel. These pipes crossed North Catamount Creek and the ridge between North and South Catamount Creeks, and discharged to South Catamount Creek immediately upstream of South Catamount Reservoir. Once the water enters South Catamount Reservoir, it can be transferred to Crystal Creek Reservoir through interconnection of the outlet systems of the reservoirs. A schematic of the Pikes Peak North Slope reservoir system is shown on Figure 1.

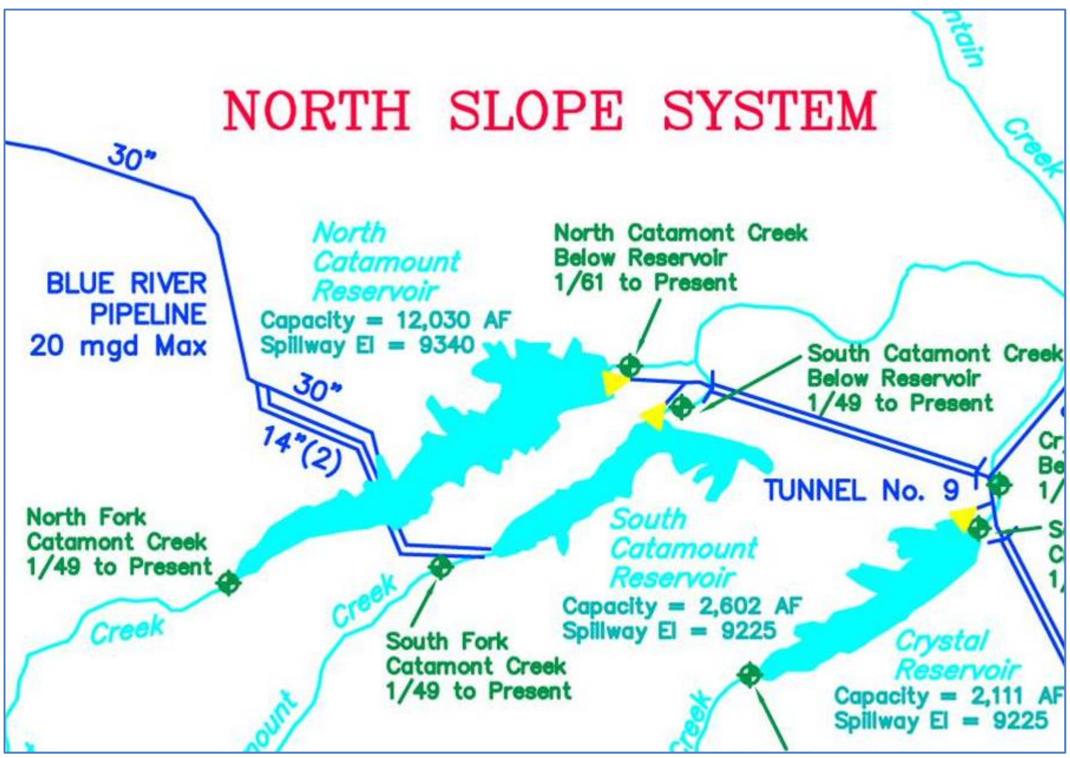

Figure 1- Pikes Peak North Slope reservoir system schematic.

In 1959, a new reservoir was constructed on North Catamount Creek to provide additional terminal storage of the Blue River water. The reservoir inundated the 14-in. and 16-in. South Catamount transfer pipes with up to 75 feet of water. An outfall flume was constructed at North Catamount Reservoir with a set of valves that allowed Blue River water to be directed to either or both reservoirs. This system worked adequately for several decades; however, with time the pipes beneath North Catamount Reservoir increasingly began to leak, until by the end of the 20th Century, Blue River water could no longer be reliably discharged to South Catamount Reservoir, and the majority of the Blue River Water ended up in North Catamount reservoir, either through the outfall flume or through leaks in the pipes. Figure 2 depicts a typical erosion feature found upstream of North Catamount reservoir created by leaks in the steel Transfer Pipes to South Catamount reservoir.

Figure 2 – Flowing water from sinkhole created by leaks in Transfer Pipe

Local water collected from the slopes of Pikes Peak typically contains high concentrations of fluoride. As a historical note, it is partly due to fluoride concentrations in water supplies of Colorado Springs and other Colorado Front Range communities that the benefits of fluoride in preventing tooth decay were recognized. Fluoride concentrations in the local water, however, exceed the limits considered beneficial to dental health, and can cause tooth staining and other adverse affects. One solution to high fluoride concentrations is to blend the local water with supplies that do not contain significant amounts of fluoride, so that fluoride levels remain below acceptable thresholds. The Blue River water is pure snowmelt water of high quality, and is an ideal blending source for managing fluoride levels.

PROJECT NEED

The importance of Blue River water as a blending source is particularly apparent in times of drought, when full utilization of all available sources of water, including water from the slopes of Pikes Peak, is critical. Following more than two decades of relatively wet climate conditions, the Colorado Springs region has been in relative drought since 2000. Drought conditions resulted in water restrictions in 2002 through 2005 and again in 2013. Leakage of the Transfer Pipe was apparent in 2002, and repair or replacement of the pipeline was discussed at that time, but no solution was implemented. During the dry years of 2012-2013, it became even more apparent that a solution to the leaking Transfer Pipe was needed. The drought had resulted in unusually low levels in North Catamount Reservoir; the half-round steel outfall flume into North Catamount was completely exposed due to low water levels. The flume was found to be severely deteriorated, and leakage from the flume eventually eroded its own foundations resulting in failure of the flume. Figure 3 shows the failed flume to North Catamount reservoir. A temporary repair to the North Catamount flume was made using corrugated metal pipe, but the need to repair the entire transfer system for Blue River water was apparent, and the decision to replace the pipe was made in July of 2013. Due to the low reservoir levels which potentially would facilitate construction and the prospect of a cold and difficult winter at above 9,300 feet elevation that would hamper construction, Utilities decided to pursue construction using a Design-Build procurement strategy. This would provide faster project development and execution than would be possible with more traditional design/bid/build procurement.

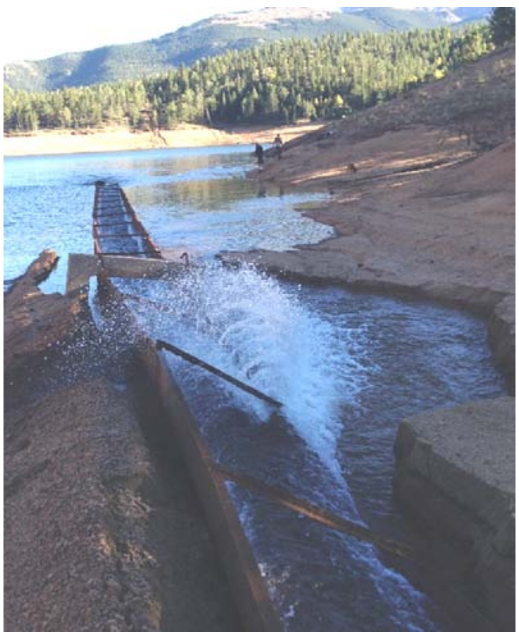

Figure 3- Failed half-round steel flume at North Catamount Reservoir

The possibility of a float-and-sink crossing using HDPE was discussed extensively by Utilities staff in developing the procurement documents for the Design/Build solicitation. Utilities has had significant experience with HDPE pipelines, primarily in the treated water distribution system, but also in the raw water supply system. As a result of this experience, Utilities technical staff was familiar with the Plastic Pipe Institute *Handbook of Polyethylene Pipe*, which includes discussion of the “float-and-sink method” for lake and river crossings in Chapter 10 – Marine Installations.

However, it seemed possible that other methods for replacing the pipeline may have been worth considering, and accordingly the solicitation did not require or even specifically identify HDPE or float-and-sink as a desired alternative.

DESIGN AND CONSTRUCTION

Garney Construction was selected as the contractor for the Design/Build effort. Garney partnered with AECOM as the engineer for the Design/Build team. While a variety of conventional alternatives were considered in the evaluation of the pipeline, it was apparent from the early stages that a float-and-sink crossing of North Catamount Reservoir was an appropriate and practical solution. The advantages of the float-and-sink crossing include the following:

- No need to drain the reservoir to facilitate construction
- Fast construction sequence in the face of impending winter
- Flexibility in vertical and horizontal alignment to accommodate subsurface irregularities
- Minimal currents would allow the pipe to rest on the bottom without cover or permanent anchorage to the reservoir floor.

Once the decision was made to pursue the float-and-sink alternative, Underwater Resources of San Francisco California was selected by Garney as a subcontractor for the floating and sinking process due to their significant project experience with this technique.

The desired discharge capacity to South Catamount Reservoir was 20 mgd. Equal capacity to the existing 14-in. and 16-in. steel pipes could easily be achieved with a 24-in. HDPE pipe; however, hydraulic analysis indicated that friction losses in the steel pipes between the HDPE section and the atmospheric vault would limit discharge to less than 20 mgd. Therefore at Utilities option a 36-in. nominal pipe size was selected. For float-and-sink applications, the dimension ratio (DR) is frequently controlled by constructability considerations. A smaller DR allows tighter curve radii for both the sinking procedure and permanent placement, and is generally more robust during handling. For this project, constructability considerations controlled, and a DR11 pipe was selected. Interior diameter of the pipe is about 30.9 inches. Subcontractor Underwater Resources provided valuable assistance in establishing a constructible and effective ballast design. Ballast was designed to provide approximately 50% buoyancy when the pipe is filled with water, allowing it to rest solidly on the bottom in addition to preventing the pipeline from overturning. Lack of currents and inability of the pipe to self-dewater precluded the need for heavier ballasting. Ballast was designed as a series of concrete blocks at 20 feet spacing placed on the bottom half of the pipe, with stainless steel straps over the pipe for

Figure 4 - Typical Precast Concrete Ballast

attachment and a neoprene sheet between the pipe and the ballast/attachment for protection. Neoprene compression blocks were placed on the anchor bolts between the straps and the anchor nuts and washers to maintain contact between the strap and the pipe during thermal contraction and expansion. Figure 4 represents a typical ballast. This is a departure from the more traditional two-piece ballast having concrete blocks both above and below the pipe, and proved to be an improvement that significantly facilitated ballast placement, and limited pipe rotation during placement.

In order to determine the most suitable alignment for the pipeline, a diver-assisted bathymetric survey of the reservoir bottom was completed. The survey identified numerous features such as an old forest road alignment, streambanks, and rock outcroppings that influenced the preferred alignment. In the final analysis, a serpentine alignment was selected to avoid discontinuities and obstructions and provided stable support conditions for the pipeline. The selected alignment is shown on Figure 5.

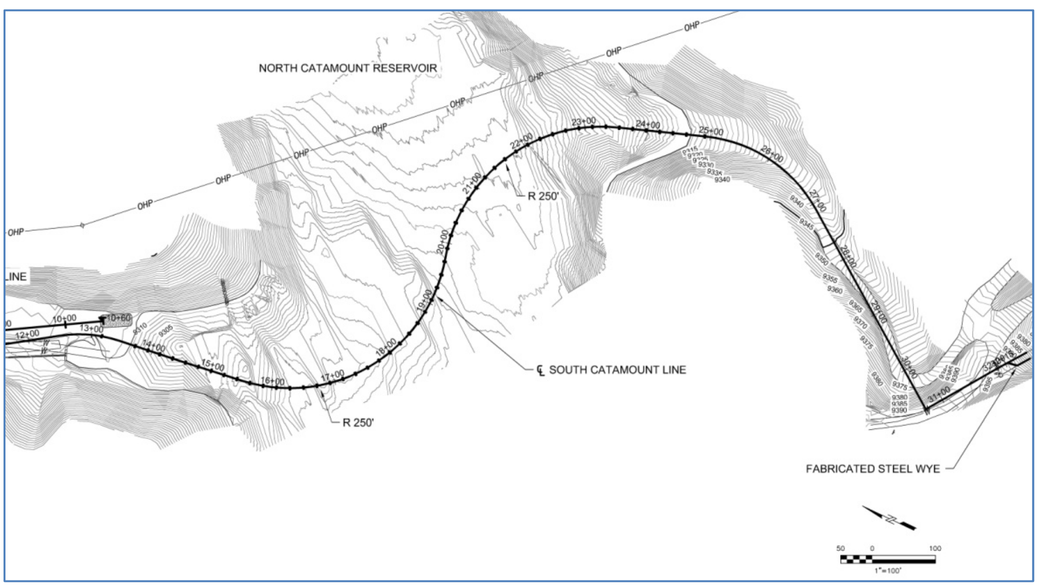

Figure 5 - Plan of alignment for float-and-sink crossing of North Catamount Reservoir.

The existing 14-in. and 16-in. pipes are adjacent to a relatively straight service road between the atmospheric vault and North Catamount Reservoir. This service road provided a staging area for fusing the HDPE pipe, allowing the entire length of HDPE to be fused as a unit prior to launching. The total length of pipe fused prior to launching was 2,450 feet. The fused length of pipe was supported on rollers to facilitate launching and held in position with mounds of earth. Figure 6 depicts a portion of the total fused length of HDPE pipe prior to floating. Watertight caps were placed on the ends of the pipe, with valves and gages installed in ports within the caps to allow eventual filling and to facilitate monitoring of the filling process. Once the fusing and capping was completed, the fused pipe was launched into the reservoir. A pit was created at the reservoir's edge for ballasting. Underwater Resources used a purpose-built steel frame in which the ballast blocks were placed, that allowed the blocks to be lowered beneath the pipe within the pit, and then raised into position against the underside of the pipe (Figure 7). The steel straps were then installed over the pipe to attach the ballast, and the pipe was advanced into the reservoir to allow placement of the next ballast block. This sequence was found to work well and allowed for rapid ballasting and launching of the pipe.

Figure 6- Fused HDPE Transfer Pipe prior to floating

Figure 7 - Ballast block installation.

Once the pipe was launched, it was maneuvered into the desired serpentine alignment and tethered into approximate position over its eventual alignment using cables affixed to onshore locations and to anchors within the reservoir. The

Figure 8 - HDPE Transfer Pipe at the start of the sinking process, floating on the reservoir surface over its eventual alignment

pipe was then sunk into its eventual location by filling with water, beginning from the north end. Figure 8 represents a photograph of the final serpentine alignment on the surface of the reservoir just as the sinking operation began. A common method for sinking the pipe involves creating a loop in the pipe by raising the beginning section of the pipe with an onshore or barge-mounted crane and filling the pipe between the raised portion and the near shore to lower the initial section of pipe. Once the initial section is sunk, the raised section can be lowered back to the surface, and as the pipe is filled with water the lowering will progress from the near shore to the far shore in a controlled manner. For this project, however, an inflatable bladder or "pig" was inserted into the pipe, and water was pumped into the pipe behind the pig. The pig was pushed through the pipe as additional water was added, resulting in controlled sinking of the pipe from the near shore to far, but without the need for a crane or other means to lift the pipe.

The float-and-sink technique was, of course, only applicable to portions of the alignment that were within the reservoir at the time the pipe was placed. Due to the low reservoir conditions at the time of placement, a significant portion of the alignment that will eventually be submerged beneath the reservoir was exposed and could be constructed in the dry. For these sections, HDPE pipe was used, but the pipe was conventionally buried in a trench, with ballasting as appropriate. In addition to the HDPE pipe within the reservoir, new pipe was installed onshore at both ends to address alignment changes and transition to the existing system. These onshore portions were made with conventional welded steel pipe. The deteriorated steel outfall flume into North Catamount Reservoir was demolished and was replaced. Due to the unusually low reservoir conditions at the time of construction, this outfall was done with 30-in. PVC placed conventionally rather than by float-sink, with ballasting as appropriate. The North Catamount outfall will be submerged in the reservoir during typical reservoir operating conditions.

The discharge of the original 14-in. and 16-in. pipes was controlled with a cluster of four valves, with cross-connections to the North Catamount outfall that allowed flows from either pipe to be directed to North Catamount or South Catamount reservoir depending on operator preference. While effective, this valving system was subject to misoperation by inexperienced personnel resulting in unintended consequences, including overflow of the atmospheric vault and severe erosion of the access road. The valve cluster was replaced with a single 30-in. plug valve located on the North Catamount outfall pipe. When this valve is completely open, all water will flow to North Catamount Reservoir. When the valve is completely closed, all water will be directed to South Catamount Reservoir. At intermediate valve settings, flow can be directed to both reservoirs simultaneously.

CONCLUSION

Since construction and putting the pipe into service in the spring of 2014, the pipeline has performed as anticipated with no issues, and operational flexibility and control with respect to the Blue River water has been restored. This project has confirmed Utilities' confidence in HDPE when used in appropriate applications, and Utilities will continue to consider HDPE for future projects where appropriate from the standpoint of constructability, performance, and economics. Based on past experience with HDPE, Utilities expects service life of the pipe to at least equal and most likely exceed that of the steel pipeline previously used. The float/sink method is somewhat specialized, and it is not clear that another opportunity to employ this technique will present itself in the near future. However, there are some situations, such as reservoir bypass conduits and outfalls where the technique could be used, and now that Utilities has had success with the technique it will be given consideration where appropriate.

Improved Design and Constructability through Five Installation Methods for One HDPE Pipeline Project

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Abstract

Most utilities are faced with replacing aging infrastructure while experiencing an ever reducing repair and replacement budget. Determining cost effective ways to repair and replace infrastructure in a cost effective manner led the South Seminole & North Orange County Wastewater Transmission Authority (SSNOCWTA) to explore multiple pipe installation methods on one project in order to reduce construction costs and disruptions to customers and businesses. SSNOCWTA replaced 8,200-feet (2,500-meters) of existing class 200 PVC pipe due to age and the high probability of failure. Pipe bursting was the first method reviewed to replace the existing force main. During right-of-way permitting coordination it was determined pipe bursting was not permitted in certain locations by the agency controlling the right-of-way. Therefore, supplementary pipe installation methods were required. Following the design process, a total of five (5) HDPE pipe installation methods were chosen. This paper will discuss the applicability and benefits of the five (5) HDPE pipe/fitting installation methods. By utilizing different installation methods, engineers can provide value to their utilities by producing a more constructible project that minimizes disruptions to customers, businesses and the environment. The additive value of all the advantages incorporated into the project saved SSNOCWTA over $215,000 and two months of construction time.

Keywords: HDPE; Pipeline; Pipe Bursting; Sliplining; Horizontal directional drilling; Jack and bore; Bypassing.

1.0 INTRODUCTION

When approaching any project, no matter the size or complexity, it is important to keep an open mind. There are different challenges and methods associated with every job and every project should be approached in a deft manner. There can be many possibilities to complete a project, but there is a special recipe that blends the ideas of the engineer, the needs of the client, and the constraints of the site. The goals and efforts contributed by Reiss Engineering (REI), South Seminole and North Orange County Water Transmission Authority (SSNOCWTA), and Murphy Pipeline Contractors (MPC) were used to make the Eagle Circle force main replacement a smooth, deft, efficient, and practical project.

The Eagle Circle force main replacement located in Casselberry, Florida, entailed five pipeline installation methods: horizontal directional drilling (HDD), pipe bursting, open-cut, sliplining, and jack and bore. The existing force main pipeline consisted of 3,950-feet (1,204-meters) 16-inch (400-mm) ductile iron pipe and 4,400-feet (1,341meters) 10-inch (250-mm) C200 PVC. The existing line was replaced based on an asset management plan that identified risk-based, high priority evaluation infrastructure. Replacement of the infrastructure was then detailed in a capital improvement plan that was adopted by the SSNOCWTA Board. The pipeline was believed to be experiencing lower wastewater flows than designed. This was verified by the existing hydraulic model. The reduced flow was the result of an existing lift station being diverted to an adjacent force main. This allowed the project to progress with a larger selection of pipe installation methods. The hydraulic model confirmed the new force main could be replaced with a continuous 10-inch (250-mm) pipe. The installation method chosen for the new pipeline was based on several competing factors: location, existing utilities, existing pipeline, roadway proximity, and easement size. By using multiple tools and accepting all possibilities, the Eagle Circle force main project was successful, completed ahead of schedule, and provided project methodology that can be used for future projects.

2.0 BACKGROUND

SSNOCWTA is an organization tasked with operation and maintenance of wastewater transmission system serving five major local municipalities in the Central Florida area including: Seminole County, City of Casselberry, City of Winter Park, City of Maitland and City of Winter Springs. A capital improvement plan is part of SSNOCWTA's planning efforts which has a schedule based on priorities for each improvement to be made. The Eagle Circle force main replacement project was created because SSNOCWTA proactively wanted to replace the existing C200 PVC and metallic pipe that was present along the Deer Run golf course and Eagle Circle due to at least four past failures.

3.0 PIPE MATERIAL

HDPE was chosen for pipe based on its versatility in the field and excellent corrosion resistance for raw wastewater service. HDPE has greater flexibility and a greater resistance to cracking or splitting when placed under compression. HDPE also uses heat fusion for joining pipe sections together, minimizing potential leakage from joints. PE4710 was selected for the grade of HDPE pipe due to its high-performance, increased flow capacity, and improved long-term performance while maintaining traditional flexibility benefits, leak-tight heat fusion joining, chemical resistance and ease of installation.

Similar to the HDPE pipe selection, pipe fitting and connections used butt fusion (see Figure 1), fused fittings and electrofusion couplings. HDPE mechanical joint adaptors were used when the new HDPE pipe was connected to the tie-in locations. The ductile iron plug isolation plug valves were the only items in the force main which did not utilize HDPE.

Figure 1. HDPE Fitting

4.0 COORDINATION

Coordination was required with the Florida Department of Environmental Protection (FDEP), Seminole County Right of Way (ROW), Seminole County Maintenance of Traffic (MOT), Seminole County Sheriff's Department, Seminole County Crossing Guard, the City of Casselberry and Sterling Park Elementary to ensure all parties were notified and had the opportunity to express concerns before the project began construction activities.

The Eagle Circle force main replacement project involved exposure to various aspects of the public. The proposed pipeline crossed a roadway, tied into an existing force main located under a high volume road, passed through a golf course, and came into close proximity with an elementary school. These various forms of potential interference called for significant amount of coordination and planning. REI, MPC, and SSNOCWTA made it a top priority to maintain constant communication with the effected parties.

Coordination with the Sheriff's Department and Seminole County was also executed early ensuring safe and successful MOT. The jack and bore portion of the project was carried out very close to the Sterling Park Elementary School. Seminole County allowed the jack and bore to be completed in the turn lane into the school, which allowed the existing sidewalk serving the school to remain in use. A line of communication was established with the school and crossing guards well in advance to further expedite the construction process which in turn kept outside entities happy and ensured safety for the students of the elementary school.

5.0 BYPASSING

The Eagle Circle project used a temporary bypass line (Figure 2) to assist with the new pipeline installation. By establishing a plan early and keeping FDEP informed, the bypass line was installed and removed with few delays. The initial plan of action for the Eagle Circle force main replacement project was to establish a bypass along the 10-inch (250-mm) pipe bursting portion along the golf course of the existing force main. The other 16-inch (400-mm) section of existing force main would be kept in service with parallel replacement. This construction method would have required the use of jack and bore at five locations. While effective, this method involved a large impact on the environment and the public in the surrounding area, and imparted a large cost to the project. In order to reduce the construction impact and cost of the project, REI shortened the required length of bypass piping and completely took the existing pipeline out of service by tying the bypass into a separate force main approximately 2,080-feet (634-meters) away. This alteration to the construction process resulted in approximately $115,000 of cost savings, helped streamline the project by taking the existing pipeline completely out of service and reduced the amount of jack and bore locations to only one location.

Figure 2. Force main bypassing

6.0 PIPE INSTALLATION METHODS

Trenchless installation methods were desired by SSNOCWTA. Pipe bursting was the first method reviewed to replace the existing force main. Pipe bursting was preferred due to larger pipe was not required and the original alignment appeared to be safe; so pipe bursting appeared to be a preferred option. During permitting coordination with Seminole County Right-of-Way it was determined pipe bursting was not permitted under Seminole County roadways. The City of Casselberry and golf course did permit pipe bursting under their roadways. Therefore, supplementary pipe installation methods were required for local roadway crossings. Following the design process, a total of five (5) HDPE pipe installation methods were chosen including: 4,400-feet (1,341-meters) of pipe bursting, 54-feet (16-meters) of jack and bore, 420-feet (128-meters) of HDD, 3,296-feet (1,005 meters) of sliplining and 180-feet (55-meters) of open cut (see Figure 3).

6.1 Horizontal Directional Drilling

The Eagle Circle force main replacement project used horizontal directional drilling (HDD) as one of its pipe installation methods. HDD was chosen based on a combination of its qualities and the site restrictions. HDD made up 5 percent of the total pipeline length, resulting in a length of 420-feet (128-meters) out of the total 8,350-feet (2,545-meters). The factors that contribute to the decision of HDD versus sliplining and open-cut are the lack of room and easement to work in the specified area, Seminole County requiring no roads be damaged during construction, and the congested utility corridor located in the area.

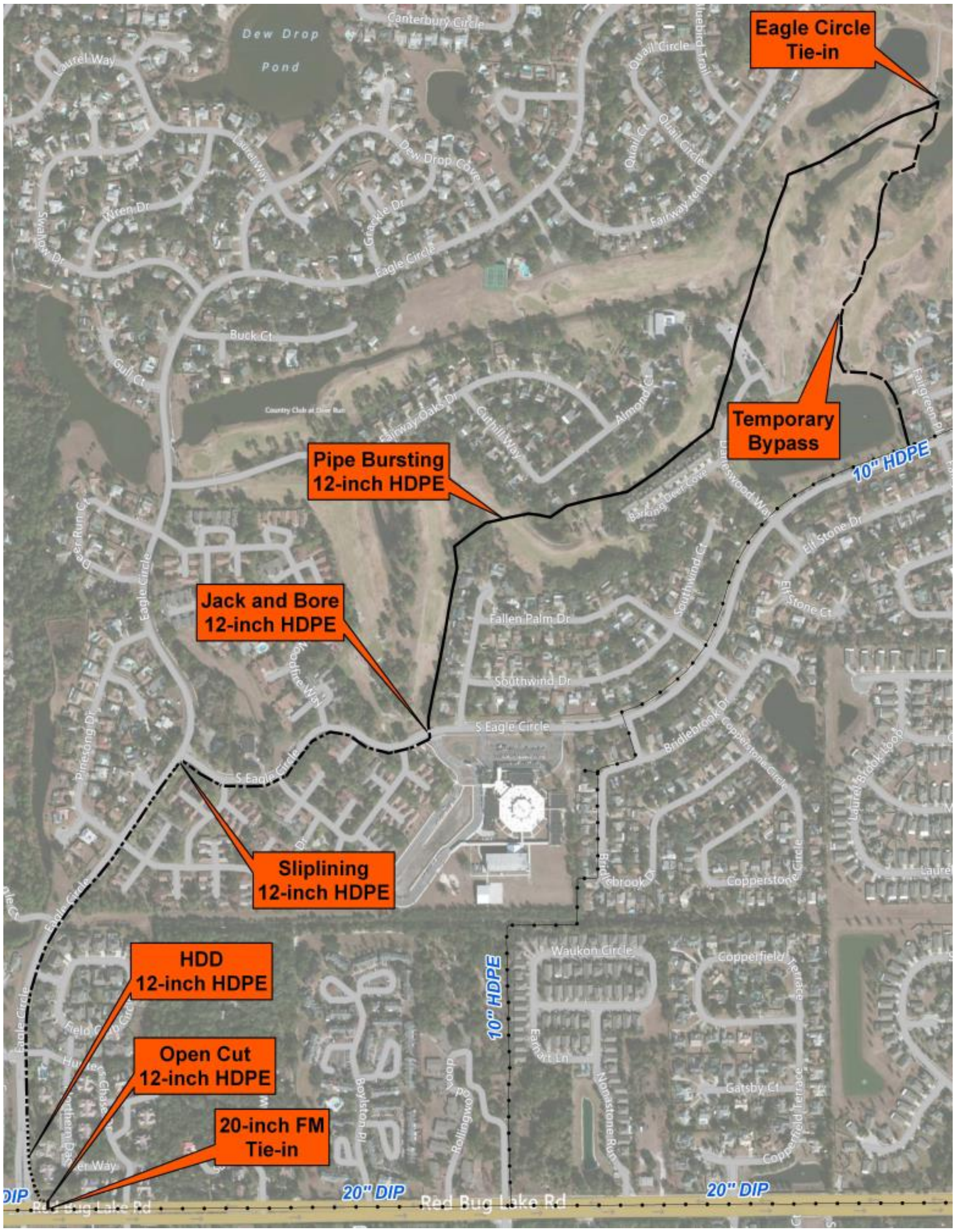

Figure 3. Replacement Pipe Installation Methods (Microsoft)

For sliplining to take place, the 12-inch (300-mm) HDPE line would have to exit the existing 16-inch (400-mm) ductile iron force main under a roadway, which would be violating the requirements of Seminole County. The chance to exploit the existing ductile iron force main was an attractive option, however, did not work due to the constraints established by Seminole County. The other potential option for the section under observation was open-cut pipe installation. The open-cut method was also not a viable solution due to the space constraints set by the size of the easement and the proximity of a masonry wall. In contrast, HDD offered pipeline installation in a confined space, allowed the pipeline to be directed under the masonry wall, and prevented the damage of roadway, damage of existing utility lines, and interference of daily roadway utilization.

6.2 Open-Cut

Open-cut pipe installation was the least attractive method to use during this project. The hesitant attitude toward this installation method stems from its potential to damage underground utilities, the large amount of manpower, the need for large equipment, and its lengthy process. Open-cut was used typically when other methods did not offer a practical, or even possible, means of installation. The open-cut sections of the Eagle Circle force main replacement make up 2 percent of the total pipeline length, resulting in 180-feet (55-meters) of the total 8,350-feet (2,545-meters).

The areas where the open-cut method was used involved a combination of tight, directional changes, plug valves, air release valves (ARV), a pit, or a fusion. The frequency of open-cut occurrences was kept at a minimum in order to save time and prevent extensive damage to above ground established landscape and roadways. Some challenges arose during open-cut installation including discovery of a gas main which was not located. MPC executed significant care during excavation activities and the unidentified gas main was not damaged. The gas main was not marked prior to the beginning of construction, even though proper notification was provided through the Sunshine State One Call of Florida (SSOCOF) call center established through the "Underground Facility Damage Prevention and Safety Act".

6.3 Sliplining

The sliplining process involves minimal equipment and digging. Sliplining is the act of passing a new pipe through an existing pipe. Typically, two pits will be dug on each of the section that is to be sliplined, and the new pipe (see figure 4) will be fed through the existing pipe with a bursting or pulling machine. This method was a great tool during the portion of the Eagle Circle force main replacement project where the 16-inch force main was downsized.

The ability to slipline was a great addition to the project as it introduces multiple benefits such as pipe structural integrity, the prevention of underground space consumption, not having to remove and dispose of the old pipe, and the reduced potential of damaging existing utility lines. The financial benefit from sliplining was the reduced number jack and bores required under existing roadways, as pipe bursting or open cut was not allowed. However, this type of pipe installation method is not possible on every job because it requires the ability to reduce the pipe size. In addition, the specific circumstances that allowed the use of sliplining solidifies the statement that every project should be approached with the fact that existing utilities can be of great value.

Figure 4. Fussed HDPE Pipe

Sliplining could only be utilized where there was an existing 16” cast iron force main. Sliplining sections of the Eagle Circle force main replacement make up 39 percent of the total pipeline length, resulting in 3,296-feet (1,005-meters) of the total 8,350-feet (2,545-meters). Sliplining is a great technique to use in situations where an existing pipe is present and is experiencing low velocity conditions and each end of the existing pipe is accessible. Sliplining was used whenever possible during the Eagle Circle project.

Considerations during sliplining included potential damage to the new pipe imparted by the existing pipe. In order to prevent this, protective linings were applied around the circumference of the existing pipe at both ends during the sliplining process (see Figure 5). A thorough and detailed assessment should be done on the health of the existing pipeline before using it during sliplining.

Figure 5. Sliplining

6.4 Pipe Bursting

A method becoming more commonly used to install new pipelines is the pipe bursting method. This pipe installation method can be beneficial to the project under consideration; however, the conditions of the project have to be correct for it to be applicable. Static pipe bursting is carried out with equipment known as a bursting machine which operates hydraulically, with rods, and a pipe bursting head. Rods are first fed through the existing pipe using the rig, which is setup in a pipe bursting pit with support columns. The rods are then connected to the pipe bursting head, which is attached to the new pipe to be installed. The new, larger, or same size pipe is then pulled by the rig through the existing pipe. During the process, the existing pipe is "burst" and left in the ground (see Figure 6). For pipe bursting to be possible during the job the existing pipe has to be the correct material, size and location to avoid heaving. The conditions present during the Eagle Circle force main replacement complimented the ideal conditions for pipe bursting with a one pipe size increase.

Figure 6. Pipe Bursting

Pipe bursting was utilized for the largest linear footage for pipe installation during the Eagle Circle replacement making up 53 percent of the total pipeline length, resulting in 4,400-feet (1,341-meters) of the total 8,350-feet (2,545-meters). The benefits of pipe bursting, when compared to the traditional open-cut method, included the speed of installation, low manpower required, the reuse of already occupied utility line space, absence of existing pipe disposal, and decreased damage to property within the construction area. The qualities of the project that made pipe bursting the "go to" method were the presence of long stretches of existing pipe, the material of the existing pipe being C200 PVC, the near absence of couplings and sharp turns, the large amounts of space to feed around 500-feet sections of new pipe, and the new pipe size being within the allowable diameter increase.

Even though there were many advantages to pipe burst during the Eagle Circle force main replacement, challenges still arose. Included in the scope of the project was a need for locator wire to be installed with the new pipe. With traditional open-cut methods, the locator wire can simply be laid on top of the new pipe and intermediately attached to the pipe. This cannot be done with the pipe burst installation method so the wire was tied to the pipe bursting head. This did not work well with the 10-gauge copper wire because the wire was becoming heavily damaged during the installation. To resolve this issue, tracer balls had to be installed on top of the new pipe near the end of the project. A larger size tracer wire could also have been pulled through with the bursting head. Other challenges that surfaced were the presence of unknown turns in the existing pipe and heaving at the surface from the new, larger sized pipe. Solutions to the these challenges involved resurfacing some patches of pavement and adding additional pipe bursting pits where sharps turns were located. Overall, the benefits heavily outweighed the challenges that were encountered for the Eagle Circle force main replacement project.

6.5 Jack and Bore

The jack and bore method first installs a casing pipe and is typically made of steel. The casing is added with equipment that pushes the casing through the ground and the dirt is removed with an auger. The new pipe is then added through the steel casing and typically filled with grout or end seal to seal the gap between the new pipe and the casing. The jack and bore installation method was the least used installation method during the project making up 1 percent of the total pipeline length, resulting in 54-feet (16-meters) of the total 8,350-feet (2,545-meters).

This method was chosen for a small portion of the job based on a constraint that was initially given by Seminole County. Seminole County would not allow any damages to occur to roadways during the project. In effect, jack and bore installation was a solution for a section of pipe that needed to be replaced. Pipe bursting could not be utilized due to Seminole County restrictions and the posed threat on the road due to heaving potential. By using the jack and bore method, a section of HDPE was installed without interrupting normal traffic flow and without damaging the roadway;

excluding the turn lane to ensure the sidewalk adjacent to the school would remain intact.

A weather condition that is very common in Florida is heavy rain. With jack and bore, the steel casing had to be welded while the casing was being pushed into the ground. The main challenge that utilizing jack and bore faced during the project was making sure the rain did not affect the quality of the welds. Portable canopies were an easy solution to this problem and should be considered if a similar situation arises during a different project (see Figure 7).

Figure 7. Welding Jack and Bore Casing

7.0 TIE-IN LOCATIONS

The new HDPE pipeline was a single, dedicated force main with no connections. One end of the new pipeline started just after a valve vault at an existing pump station located at the north east end of the golf course, and the other end tied into an existing force main located under a high volume County road. Tie-in of the new HDPE involved MOT, installation at night, and removal and restoration of roadway.

Connection to the existing lift station was done through a 12-inch (300-mm) by10-inch (250-mm) reducer and a section of PVC. Additional underground piping was replaced at the pump station due to a condition observed during excavation. The other end of the new pipeline was tied into an existing 20-inch (500-mm) ductile iron force main located under a heavy flow County roadway. The connection was possible through an existing, operational tapping gate valve positioned horizontally. To ensure a successful install, REI and MPC collaborated with Seminole County and the Sheriff's Department to setup MOT. A preliminary night excavation was completed at the gate valve in the road to ensure the existing gate valve was operational. A two-inch test corporation stop was also installed to ensure the gate valve was completely isolated. During the tie-in process a pump truck was on site to remove any remaining solids and liquids. Plastic was installed under the existing force main to prevent wastewater from leaching into the soil (see Figure 8). The use of the existing gate valve saved SSNOCWTA from being required to install a new tapping saddle, tapping valve and additional road excavation.

Figure 8. Red Bug Lake Road Existing Pipe at Tie-in Location

8.0 CONCLUSION

The Eagle Circle force main replacement project was successful thanks to the collective efforts of SSNOCWTA, MPC, and REI. The project was completed ahead of schedule, experienced little delays, and imparted no unintended damage to the existing site. The use of multiple pipe installation methods reduced the cost of the project, created less potential for existing utility line damage, and incorporated several benefits to the project.

The execution method used during the Eagle Circle project created many benefits for everyone involved including SSNOCWTA, REI, MPC, and the general public. Advantages that were incorporated into the project range from cost savings to reduced construction schedule time while minimizing damage to third party utilities and the environment.

The Eagle Circle project incorporated an advantageous bypass route that reduced the amount of jack and bore locations required and amount of bypass piping required. The tie-in location located under the roadway was completed using an existing gate valve rather than having to carry out a wet tap which avoided additional road damage. During pipe bursting several repair clamps were encountered. Proper planning and coordination between FDEP, Seminole County ROW, Seminole County MOT, Seminole County Sheriff's Department, Seminole County Crossing Guard, the City of Casselberry and Sterling Park Elementary ensured a smooth project. Incorporating trenchless technologies such as pipe bursting, sliplining, jack and bore and HDD reduced roadway damage, golf course disruption, and involvement of nearby residents. The additive value of all the advantages incorporated into the project saved SSNOCWTA over $215,000 and two months of construction time. Upon completion of the project, it was found that using existing infrastructure, applying multiple pipeline installation techniques, and keeping a constant line of communication between involved parties can save the client costs and reduce completion time.

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CSO Projects—What Is the Right Solution? A Case Study for South Bend, Indiana

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Abstract

The City of South Bend's Consent Decree specifies capture and conveyance of overflows from all of their Combined Sewer Overflow (CSO) locations, which was issued in 2012. The Long Term Control Plan (LTCP) in the Consent Decree, which was developed from 2002 to 2008, has a cost of almost $600 million. There are several retention treatment basins (RTB) and storage/conveyance pipes planned in the Long Term Control Plan (LTCP) as part of the Consent Decree. After the City of South Bend went through the first phase of their LTCP, they found that affordability was an issue. At the same time, the City was completing a feasibility study which ultimately included looking for options to reduce the cost of a specific area of the CSO program based on new technology. As a result of finding significant savings in that study and with the recent development of integrated planning, the City elected to contract a consultant to reevaluate their Consent decree LTCP. This paper describes the process for re-evaluating LTCPs through the use of new design technologies, integrated planning, and the value of advanced affordability calculation methods and how it is being applied to find the right answer for the City of South Bend.

I. Introduction

As this country continues to deal with a tough economy with continually escalating costs for basic utility services, especially being more difficult on the lower and average income citizen, there are several hundred communities that face the added challenge of having to fund an expensive program to reduce combined sewer overflows (CSO). We all want to improve our environment and one of the tasks at hand to do that includes reducing the number of combined sewer overflows going to our surface waters. However, the question that comes with that responsibility is, at what cost is the right solution?

1. CSO Regulation and Design History

A. EPA Policy

In 1969, after another of several river fires in the Cyuyahoga River in Cleveland, Ohio, the federal government formed the Environmental Protection Agency. One of their first objectives was to prevent further pollution of our waters which resulted in the inception of the Clean Water Act in 1972(1). This document gave the Environmental Protection Agency (EPA) the ability to take on the fight against water pollution with the regulations to control discharge of pollution. Initially, this fight included stopping direct discharges from public and private wastewater facilities as well as unregulated discharges. It took several years and billions of dollars, but we have a come a long way to eliminate those types of pollutant sources. Fortunately, a lot of the improvements on the public side were funded by local, state, and federal agencies, which made the burden of the cost easier for all citizens.

In 1989, the EPA took environmental issues a step further and published the "CSO Control Strategy" (2) which established CSO control based on technology and water quality. This document outlined two approaches for the approvable technical methods for this control and communities were expected to achieve desired environmental benefits based on these two approaches. Then, in 1994 the EPA the published their "CSO Policy" (3) document which provided their understanding of cost effective CSO Controls. Within that document, there are two phases that were to be incorporated for CSO control. Phase I, titled Nine Minimum Controls, is about operation and maintenance procedures to achieve a beneficial level of results without spend a lot of money. The second phase is for the communities to develop long term control plans that generally would require significant funding.

B. Historic CSO Control Design Approach

For the communities that have proceeded with their CSO program as far back as the 1980's, most of their long term control plans included two different types of control solutions. They are by sewer separation or by storage.

i. Separation

The sewer separation solution is simple in concept and obviously very effective as a new collection system is built and the storm and sanitary flows are separated so the system is no longer combined. Most of these were done in small communities that did not have a lot of downtown areas and sewer separation was proving to be cost effective. Larger communities with extensive downtowns typically would not select sewer separation as the cost and disruption to excavate those streets often made that solution unfeasible.

ii. Storage

The engineering approach behind the design of storage systems has historically been to model the collection system, calculate the volume of overflows at the point of discharge from the CSO outlets, then size the tunnels or retention treatment basins for those volumes. These systems have proven to be effective through monitoring programs, but they were expensive to build. The storage solution is where tunnels or retention treatment basins are installed to collect the overflow volumes, store them, and treat them to a certain level before they are either routed back to a wastewater treatment facility or allowed to be discharged to surface waters.

2. South Bend CSO Program Status

The City of South Bend is just one of almost 800 CSO communities across the country dealing with LTCPs. Many are unaffordable and pose multiple social and economic impacts to their residents. The South Bend CSO system has 36 CSOs which typically overflow 60-70 times each year, resulting in approximately 1B gallons of overflows into the St. Joseph River. In addition, when the CSO system's capacity is exceeded, basement backups can occur.

Historically, the City invested in wastewater infrastructure to reduce CSOs prior to the formalization of a LTCP or signing of the consent decree. During the period between 1990 and 2004, the City spent over $87M on CSO-related projects; thus, implementation the first phase of the eventual LTCP.

In addition to the those CSO type projects, the City was motivated by the idea of doing more with less, and focusing on getting as much capacity from the current collection and wastewater system as possible. With that, the City deployed approximately 120 real-time monitoring sensors throughout the sewer collection system in 2005. This was an attempt to answer one simple question: Is the City making the most of the existing infrastructure? In other words, if there is unused capacity within the existing collection system (i.e., within the main interceptor sewer or at the WWTP), and how can this best be utilized?

While the City was going forward with LTCP projects and after a considerable amount of negotiations, the City of South Bend, Indiana and the U.S. Department of Justice finally entered into a consent decree relating to the Clean

Water Act CSO Strategy in early 2012. The LTCP document was created in an effort to record the historical context of the evolution of the plan, as agreed to in the consent decree.

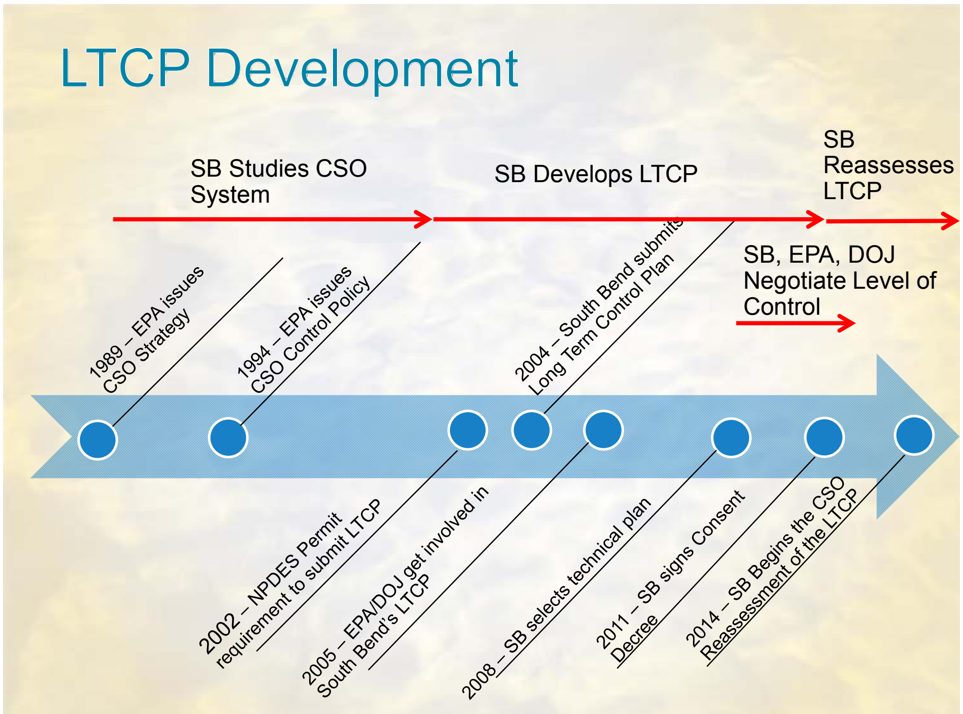

Figure 1 South Bend LTCP Development

The 2012 LTCP includes wastewater treatment plant (WWTP) upgrades and CSO control methods, separated into two phases of work within the sewer collection system. Phase 1, includes source control technologies to prevent storm water from entering the collection system. Phase 2 consists of the ultimate conveyance of excess flows from the existing CSOs to nine separate storage facilities.

As allowed for in the consent decree language, the City has continued to look for ways to improve upon and reduce LTCP costs. The City recently investigated the feasibility and potential cost savings of using a combination of low impact development, real time control, and conventional methods for CSO control. The resultant report from that Optimatics Study (4) was a feasibility and planning document that provided useful insight for optimization of the 2012 LTCP.

In addition, several improvements have been made to the CSO system beginning in 2010 to 2013 as a result of real-time monitoring, that have reduced runoff into the combined sewer area. Improvements included reducing combined sewer area via separation projects, raising weir heights, and throttling more flow to the main interceptor sewer to reduce overflows.

In 2014, tremendous outcry from a new City Council, as well as numerous concerned City ratepayers, led the City to initiate a reassessment of the 2012 LTCP in order to find additional cost savings in the realm of at least \$100-200M. The reassessment of the 2012 LTCP will review system optimization and evaluate several options based on real-time technology. For example, a preliminary review of real-time data has identified that potential trunk line interconnections throughout the collection system may move flows from satellite areas of the system to the WTTP, eliminating the need for possible storage as planned in the 2012 LTCP. Opportunities for inline storage and regulating flows at other CSOs to the interceptor will also be evaluated. The reassessment of the LTCP will follow a comprehensive approach to include integrated and cost-effective solutions that also consider elements of low impact development and green infrastructure.

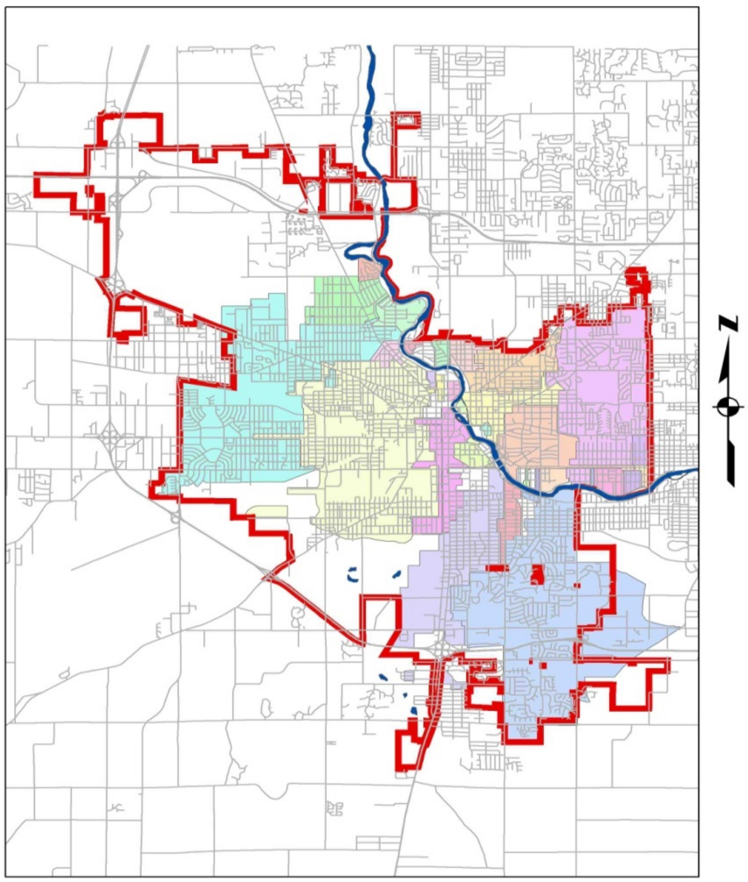

**Figure 2 South Bend CSO Service Areas:
20 Square Miles
(13,069 Acres)**

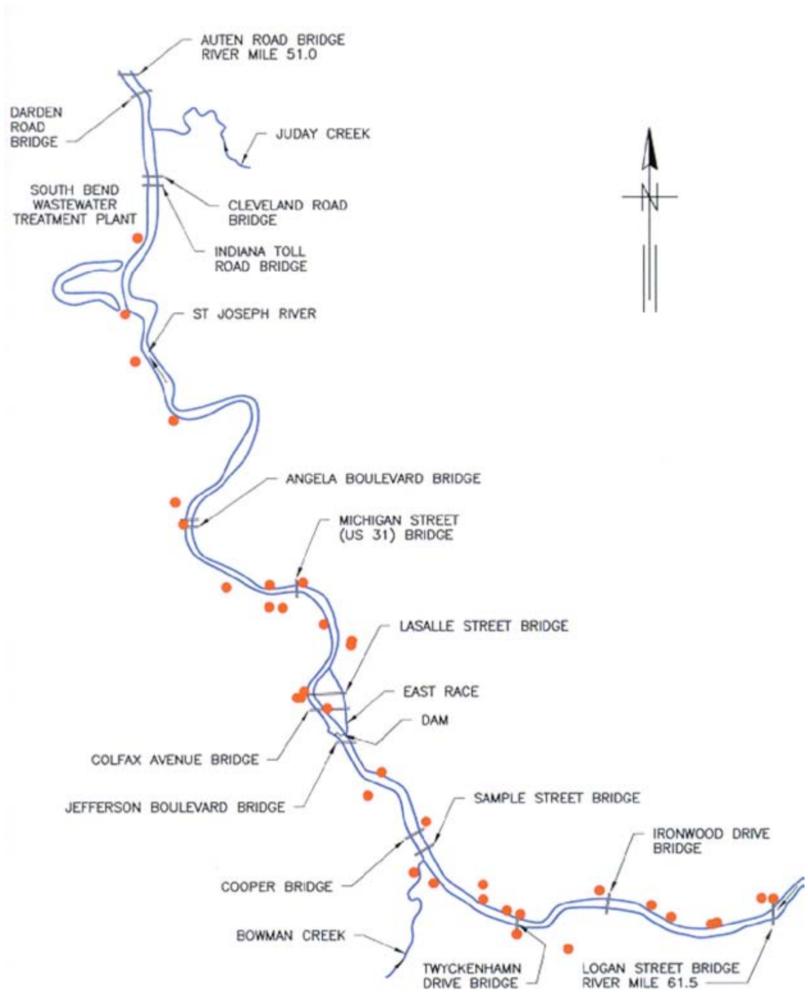

Figure 3 South Bend CSO Outfalls

3. South Bend Riverbank Stabilization Study (5)

In 2013, the City of South Bend took on a study that included three objectives within a select area of the City along the St. Joseph River. They were to stabilize the riverbank, construct a trail way for pedestrian and non-motorized vehicles, and install the conduit planned for CSO control in that area. The CSO portion of the study included evaluation of CSO 007, 008, 010, 011 A and 011B, which are located on the south side of the river between Lafayette Boulevard and Angela Boulevard.

A. Current LTCP Design for the Riverbank Study Area

The study area was in one of the South Bend CSO sub-districts, was located on the north side of the City, and is commonly referred to as the "Leeper Park" area. The design of the improvements for this area is based on the traditional CSO control method for end of pipe capture and included the following facilities:

- 12,000-ft conveyance and storage conduit from CSO 006 to the WWTP (12-ft diameter at the WWTP)
- 1.0-MG storage facility at Brownsfield Park
- 8.7-MG storage facility at Leeper Park and associated consolidation sewers
- 5800-ft conveyance and storage conduit on the East bank (12-ft diameter)

The facilities at each of the storage tanks and East Bank conduit include screening, a dewatering pump station (to dewater contents through the existing interceptor to the WWTP following each wet weather event), provisions for solids removal, and emergency overflow with disinfection. Figure 4 shows the current Leeper Park sub-district LTCP projects.

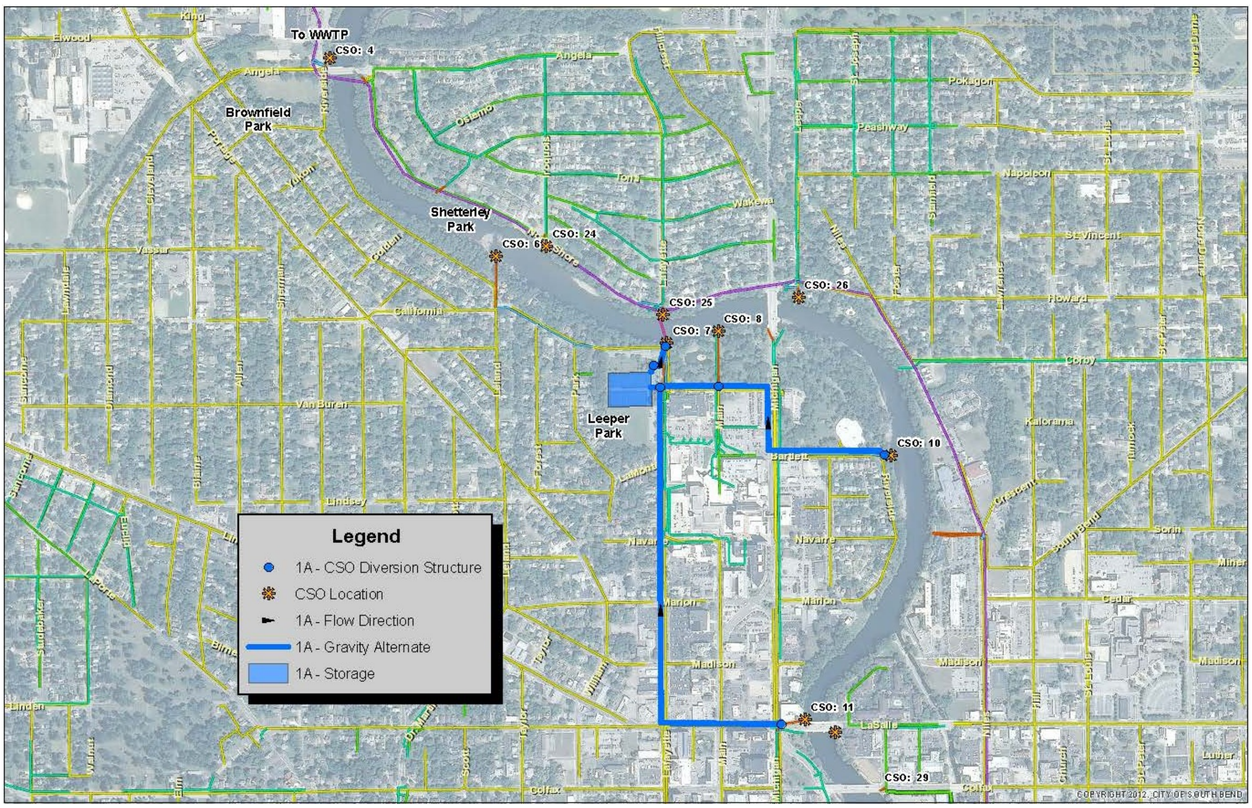

Figure 4 Current LTCP Design - Leeper Park Sub-District

B. Riverbank Study Alternates for CSO Control

During the initial evaluation of alternatives to route these CSO overflows to the treatment plant along the river, it became evident it would beneficial to progressively look at CSOs further from the study area with ideas to consolidate additional storage and transportation. These ideas included modification to the planned 12,000-ft conveyance and storage conduit from CSO 006 to the City's Wastewater Treatment Plant (WWTP) and 5,800-ft conveyance and storage conduit on the East bank (12-ft diameter) and possible size reduction of the two RTBs in the system.

As the study progressed it became evident that a new plan for the CSO control might be able to save the City considerable costs using an optimization approach.

Several alternates were then analyzed and conceptual estimates prepared for their comparison to the current LTCP. The total costs for the alternates are as follows:

Original Plan	**$213,256,000**
Alternate 1A	**$201,190,000**
Alternate 1B	**$196,640,000**
Alternate 1C	**$179,470,000**
Alternate 2	**$220,970,000**
Alternate 3	**$155,130,000**

Alternate 3 is based on a capture plan that utilizes existing pipes in the "real time control" and "optimization" approach that was able to eliminate the two storage basins planned for this area and save considerable costs for the program. Figure 5 shows the alternate 3 layout of the CSO control system for the Leeper Park sub-district.

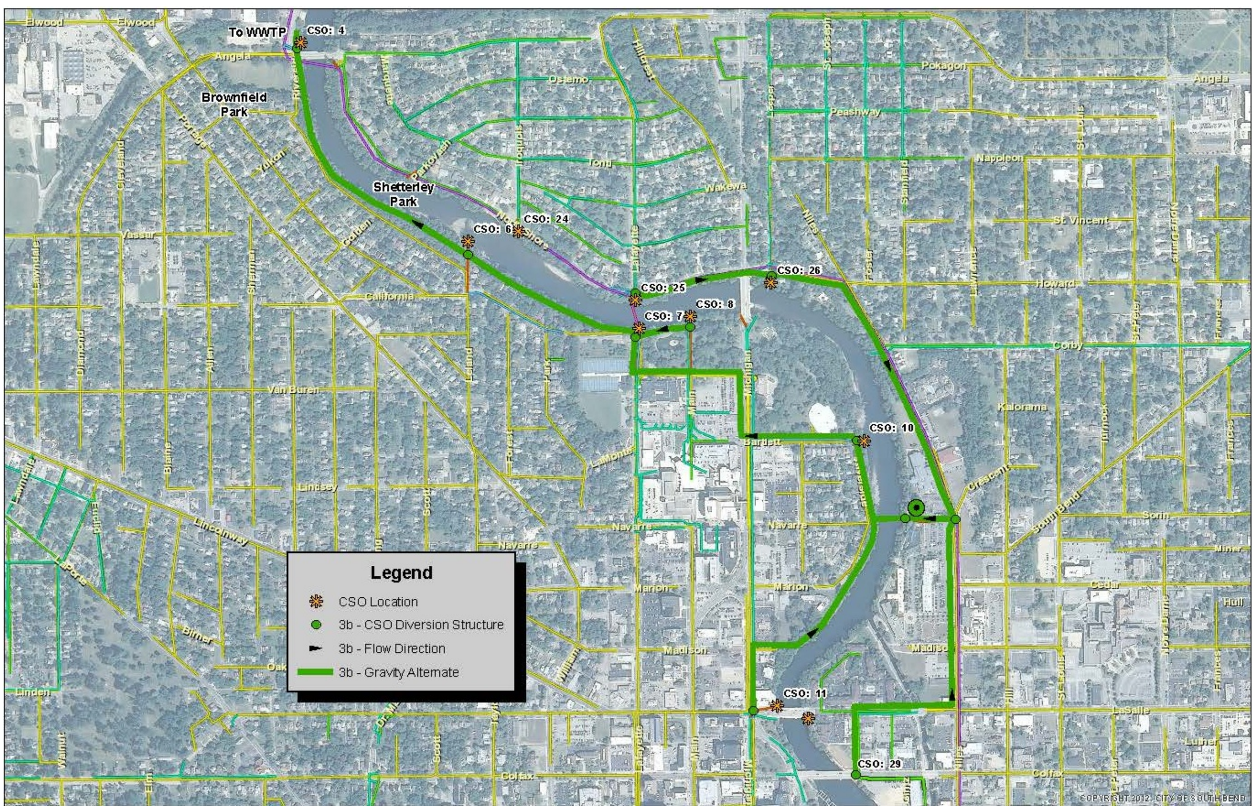

Figure 5 Alternate 3 CSO Control System - Leeper Park Sub-District

4. Affordability Approach

A. New Approach to Program Costs for Affordability

i. Optimization

As communities proceed with their planning and design of projects for their CSO programs, it is becoming more common that the design teams use two tools for developing the programs for their clients. The first tool, the optimization approach for design of CSO volumes has proven to be beneficial and very cost effective, as shown in the South Bend example. Of course, we must be very conscientious for consequences that may result in using collection systems for storage such as basement backups, but the end result can be accomplished with proper planning of overflow elevations and control of hydraulic grade lines.

ii. Integrated Planning

The other tool for developing affordable plans is the Integrated Planning Framework (IPF) for financial capability assessments that determine the financial burden on the rate payers. This is a financial tool that allows the community to use CSO project costs and all costs associated with the Clean Water Act, such as storm water projects, operation and maintenance, and replacement costs. Also, in some cases, while the EPA IPF guidance emphasizes Clean Water Act-related expenditures associated with wastewater and storm water needs, the IPF can also recognize that utilities may need to consider water-related expenditures under the Safe Drinking Water Act for long-term utility sustainability. In addition to including other related utility costs as a factor in affordability, the detailed analysis has been expanded to consider more thoroughly different income levels in the community as opposed to just median household income.

One more beneficial development in recent years for approval of more affordable plans has been the allowance of longer schedules to complete the CSO program. This is a key to affordability. Historically, approved CSO programs were limited to schedules up to 20 years as stipulated in the EPA 1989 CSO Strategy. Now, due to the recent downturn in the economy and increased pressure from local governments recognizing the burden for the rate payers, the EPA will consider more than 20 years.

5. Conclusion

In conclusion, there are ways to make our dollars go further for CSO programs. With current technology and advanced design techniques for optimization of existing collection and treatment system combined with the use of integrated planning, communities are finding more efficient solutions to achieve required water quality benefits that satisfy regulated CSO control measures. Even though the City of South Bend has not completed their re-evaluation of the CSO LTCP, there appears the right answer with new technology design solutions and integrated planning will make a difference to the cost for the rate payers.

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Deep Water Coastal Stormwater Outfalls: Designing for the Surf Zone

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Abstract

The City of Myrtle Beach has become one of the number one beach vacation destinations in the United States. Known as the Grand Strand, Myrtle Beach hosts over 15 million beach and golf visitors each year. For several years, the City had been plagued by adverse publicity associated with high bacteria counts in the surf zone. Recommendations were made for replacement of many beach outfall pipes and combining the outfalls into a single deep water ocean outfall. Successful implementation of the Ocean Outfall master plan was imperative because stormwater treatment upstream of the existing beach outfalls must be designed, permitted and installed to justify combining the individual beach outfall pipes. Water quality discharge requirements are identified and best management practices discussed as related to the design of the deep water ocean outfall. Pipe hydraulics, including dynamic wave analysis, is presented. Critical design elements presented are evaluation and qualification of pipe material types, jointing methods, pipe bedding, backfill and armor stone sizing for proper performance within the surf zone.

BACKGROUND

The Downtown Redevelopment Corporation (DRC) was created in 1999 by the Myrtle Beach City council and has been charged with the task of redeveloping the downtown area of the City of Myrtle Beach, attempting to reverse the decline of the quality of the neighborhood, with the goals of turning the area into an inviting year round tourist destination. The DRC's vision for the revitalizing of the downtown is to create a "contemporary identity and sense of place based upon its traditional and historical key attributes and values".

The Downtown area of Myrtle Beach had been experiencing a significant decline in tourism since the closing of the Pavilion Amusement Park in 2006. This has occurred while the tourism numbers for the remainder of the Grand Strand have remained constant in spite of the poor national economy. In an effort to increase the tourism in the downtown area the Myrtle Beach Boardwalk was opened in 2010. The impact created by the construction of the 1.2 mile Boardwalk has been phenomenal, greatly surpassing the hopes and goals of the City, the Chamber of Commerce and downtown merchants. The opening of this attraction brought more than 100,000 people to the Boardwalk area in 2010, generating an interest in the Downtown area from locals, as well tourists.

The DRC area covers approximately 370 acres of land which is roughly defined as the area located from Kings Highway to the Atlantic Ocean and from 6th Avenue South to 16th Avenue North. The City of Public Works Department, due to their responsibility for the infrastructure components is involved in this effort to redevelop the downtown area of the City. In order to facilitate this process, key staff members determined the need for a comprehensive study of the infrastructure in the DRC area. As a result of this, a stormwater master plan of the 4th Avenue North drainage basin was commissioned.

EXISTING INFRASTRUCTURE

The 4th Avenue North drainage basin is also known as the southern DRC area and is generally defined as the 90 acre area north of 1st Avenue South to 9th Avenue North and from the Atlantic Ocean to Chester Street. Chester Street being the western boundary of the southern DRC study area. DDC was commissioned by the City of Myrtle Beach to develop a stormwater management plan for the southern DRC drainage basin. The DRC area is a highly developed section of the City of Myrtle Beach. Existing development in the southern portion of the DRC area includes the former Pavilion Amusement Park area, a commercial district between 8th and 9th Avenue North as well as numerous hotels and motels throughout the area with a few beach houses. Most of the area was developed prior to current stormwater management practices.

Currently, there are eight (8) individual beachfront outfalls that drain the Southern DRC area. These pipes discharge above the high tide line and travel across the beach into the surf zone of the Atlantic Ocean.

These outfalls are unsightly to the tourists who are attracted to the beach, undesirable to the surroundings, contribute to beach advisories regarding high bacteria levels in the surf zone after significant rainfall events, and often erode large amounts of sand from the beach which must be re-nourished by City personnel. It is the City's goal to eliminate as many of these outfalls as possible. Unfortunately this is a significant cost in the removal of these outfalls due to the upstream improvements that are required to re-route the stormwater to a common discharge point. The only practical alternative to the current beach outfall pipes is to install deep water ocean outfalls which discharge the stormwater more than 1,000 feet off-shore.

REGIONAL SYSTEM

The primary goal of the City of Myrtle Beach Downtown Redevelopment Committee is the redevelopment of this study area. From a stormwater perspective, this means two things: existing undeveloped areas may be developed, increasing the amount of stormwater runoff, or existing developments may be replaced with developments that utilize stormwater Best Management Practices (BMP's), which will have a positive impact on the quality of the stormwater runoff.

A regional stormwater concept will work well in the southern portion of the DRC area. The regional concept takes the entire study area as a single drainage basin, with each individual lot discharging stormwater runoff into this drainage system. With this approach, a certain amount of stormwater runoff can be allowed for each parcel in the area, without having to know exactly how the lot will be developed in the future. This is accomplished by designing the stormwater system with a certain amount of impervious area as a percentage of total area, calculated for all the available land in the entire drainage basin.

This regional stormwater system has been designed to handle the stormwater runoff from the entire area with a 85% impervious area ratio considering each lot and public street in the basin. Actual impervious areas may vary, but the City can require that proposed developments in this area not exceed 85% impervious as a percentage of total area.

Existing stormwater runoff throughout the study area drains to several beachfront ocean outfalls located above the high tide line. This method of stormwater discharge has been the norm for all coastal communities. Since the beach is the main tourist draw to the area, these beach outfalls have become a liability to the City of Myrtle Beach. Beachfront ocean outfalls affect the beach in several negative ways. When there is no rain, the outfalls detract from the aesthetics of the beach, reduce available beach to tourists looking for an open area of sand, and often collect pools of water at their openings that attract birds and children who play in the unsanitary water. During a rain event, the beach outfalls perform their job of discharging stormwater, but also erode the beach, creating a 'mini-swash' from the outfall to the ocean. Often, these mini-swashes get quite large and have to be filled in by City personnel.

With the stormwater discharging further out in the ocean, the negative impacts to the important surf zone are reduced. Obviously, the farther away from the beach the stormwater is discharged, the more diluted it will be by the time it reaches the surf zone. Also, the waterborne bacteria and other disease carrying organisms that are measured by South Carolina Department of Environmental Control (SCDHEC) to gauge the safety of the ocean waters cannot survive for an extended period in a saltwater environment. By the time the stormwater has mixed with the ocean water and reaches the surf zone, significant quantities of the disease carrying microorganisms will have been dispersed and destroyed, so they will not have a level of concentration high enough to pose a significant threat to human health.

FLOWS

A deep water ocean outfall was recommended for the street-end at 4th Avenue North, which is approximately the mid-point of the southern DRC drainage basin. It was determined that parallel 84" pipes will be necessary to be installed to carry the 50-year storm event 1,100 linear feet off-shore in accordance with the results of the plume study which was completed and approved by the agencies from one of the previous outfall projects. A 50-year 24 hour storm event with 8.6 inches of rainfall was used to evaluate the proposed collection system for flooding.

Table 1 - Total Basin Stormwater Runoff Quantity

Storm Frequency	Rainfall Amount	Pre-Development Runoff
2 year	4.3” / 24 hours	265 cfs
5 year	5.7” / 24 hours	380 cfs
10 year	6.7” / 24 hours	406 cfs
25 year	7.6” / 24 hours	492 cfs
50 year	8.6” / 24 hours	506 cfs
100 year	9.7” / 24 hours	536 cfs
*100 year	4.1” / 1 hour	522 cfs

**This shorter duration storm is more typical of the intense summer thunderstorms that cause many flooding problems in the City of Myrtle Beach.*

PERMITTING

The 4th Avenue North outfall permit was unlike the previous outfalls permitted along the South Carolina coast. The first outfall constructed by the City of Myrtle Beach was permitted through the Charleston District of the U.S. Army Corps of Engineers (USACE) under their “Nationwide” Permitting Program. A Nationwide Permit #12 was issued for the project along with permits from SCDHEC and U.S. Coast Guard. The second outfall project was permitted under two (2) Nationwide Permits # 7 and 12 along with SCDHEC and the U.S. Coast Guard and the third outfall project was permitted under three (3) Nationwide Permits # 7, 12 and 33 again with permits from SCDHEC and the U.S. Coast Guard. Needless to say, the permitting process is getting more difficult each time an outfall was submitted for permitting.

<u>NATIONWIDE PERMITS</u>

#7 Outfall Structure and Associated Intake Structure
#14 Utility Line Activities
#33 Temporary Construction, Access, and Dewatering

So, when the 4th Avenue North outfall came up for permitting the USACE ruled that the project was ineligible for their Nationwide Permit like the other three (3) previous Myrtle Beach outfall projects were. As such, the 4th Avenue North outfall project applied for a “general permit”. A General Permit requires the input and approval of four (4) federal agencies, as well as four (4) state agencies. In the State of South Carolina, no federal permit can be issued in the coastal zone without state certification. Below is a list of the agencies involved with the permitting of the project.

FEDERAL

- Environmental Protection Agency (EPA)
- U.S. Fish & Wildlife Service
- National Oceanic and Atmospheric Administration (NOAA)
- United States Coast Guard

STATE

- S.C. Department of Health & Environmental Control (SCDHEC)
- S.C. Department of Natural Resources (SCDNR)
- S.C. Historic Preservation Office (SCHPO)
- SCDHEC Ocean and Coastal Resource Management (SCOCRM)

EPA at first ruled that the project should be reviewed under the regulations from Section 401 of the Clean Water Act, but after twenty-four (24) months of meetings, reports, and letter writing with EPA, USACE, and SCDHEC representatives, it was determined that the proposed 4th Avenue North outfall would not be a detriment to the Atlantic Ocean under Section 401 of the Clean Water Act. So, a permit for construction was issued by the agencies for the project. There were only a few conditions added to the permit by SCDNR for special accommodations to make sure that the project would not have an adverse effect on Sea Turtles during nesting season between May 1 and October 31.

WATER QUALITY

Equally as important as the ability of the system to handle the quantity of stormwater runoff, is the quality of that water as it leaves the system. There are numerous factors that contribute to pollution in the stormwater runoff, and almost as many options available to treat it.

The biggest water quality problem in the study area is litter and other debris that is swept into the stormwater system. The residents and businesses in the area certainly contribute some of the litter, but tourists and other non-residents leave the majority of the litter throughout this area. This problem is harder to control since visitors to an area, no matter how beautiful the area, rarely treat it with the respect they show to their own neighborhood.

Education and public outreach programs will help reduce the problem, but the stormwater system must be designed to separate and detain as much of this material as possible. As unsightly as it is on the side of the street, it is a potential liability on the beach or in the ocean.

The City currently operates a variety of cleaning programs to minimize this problem. Street sweepers regularly clean the roads and gutters throughout the study area, which picks up and disposes of a significant amount of trash before it enters the drainage system. Also, the City has a fleet of beachcombing vehicles which regularly pick up and dispose of trash left on the beach before it gets washed into the ocean or causes an injury to someone on the beach. City crews also collect and dispose of all the trash collected in the numerous trash cans located throughout the area. Unfortunately, even with properly spaced trash receptacles, some individuals will still drop their trash on the ground rather than dispose of it properly.

Numerous catch basins and perforated pipes in this area contain a large amount of rubbish and debris, despite the best efforts of the City to remove the debris before it has a chance to get into the drainage system. Any new storm drainage system proposed for this area must include provisions to separate and detain this refuse so that the City can dispose of it properly before it gets discharged into the ocean.

A header piping system was constructed in the property owned by the City of Myrtle Beach adjacent to the beach that runs between 8th Avenue North and 1st Avenue South under the new Myrtle Beach Boardwalk. This header piping system would be used to collect all of the stormwater runoff from the upper drainage basin reaches and transported the stormwater to the outfall at 4th Avenue North. This system would be considered a clean system, meaning all of the stormwater discharging into the system would run through a series of gross pollutant BMP devices prior to entering into the header piping system. Then the header system would provide the final stormwater treatment by coming in contact with the saltwater in the piping system which fluctuates with the tide. These tidal actions aide in the mixing of the stormwater with the saltwater to provide the necessary contact time which aides into the destruction of any remaining harmful bacteria prior to being discharged into the Atlantic Ocean.
In order to capture litter and debris prior to the stormwater discharging into the outfall header piping system, the project included installation of 8 water quality treatment vaults. These vaults were located inland of the beach zone and constructed below grade at the terminal ends of the avenues that overlie each of the eight (8) header tie-in outfall pipes. Each vault contains 3 linear radial debris capture screens that are designed to capture all debris 5 mm and larger. The stormwater passes through the screen's interior and exits through the louvered openings. The captured debris is accessed through the top hatchways and is removed using standard vacuum equipment. The screens are manufactured of Type 316L stainless steel and each measures 24 inches in diameter and 15 feet in length. Each vault is designed to treat up to 80 cfs. Inspection of the vault that was installed and connected in 2010 reveals the screens are operating as expected.

It has been found that source treatment is the most effective way to treat large urban drainage systems. The runoff flows and velocities are too high to treat the stormwater at a single point. When it comes to litter, debris and other floatables they are easily re-suspended during high flows. The addition of several treatment trains devices method reduces the overall re-suspension of the gross pollutants. The combination of hoods and deep sump catch basins along with catch basin inserts will greatly reduce gross pollutants and sediments. The radial debris capture screens were add to the system as a final source to catch any remaining gross pollutants that were either in the stormwater that were missing by the other devices or was re-suspended by high flows.

The deep water outfall projects reduce pollution, reduce beach erosion, improve water quality, improve aesthetics, provide a fish habitat and provide the fifteen (15) million visitors with the best possible beach experience.

LESSONS LEARNED

There is no one magic bullet when it comes to removal of gross pollutants, such as sediments, oils and greases, plastics, floatables, cigarette butts, etc. The gross pollutants are easily re-suspended in higher flows and can be washed through the almost all BMP devices or through their overflow bypass systems. It takes source treatment BMP's and BMP treatment trains to effectively reduce or eliminate all of your gross pollutants. It is nearly impossible to treat large drainage basins at a single discharge point especially in dense urban areas where large volumes of gross pollutants are present and will diminish the capacity of any BMP.

SURF ZONE PIPELINE HYDRAULICS & DESIGN WAVE ANALYSIS

The South Carolina shoreline is vulnerable to category 5+ storms during the 50 year storm event required for deep water ocean outfalls. Once the geographical location of an outfall is defined, the forces upon the pipeline can be calculated. The design wave and stability of the buried pipeline against pullout from wave force dynamics are evaluated and appropriate bedding, trench design, armor protection and pipeline materials selected.

For the Myrtle Beach area the 50-year design wave has a period of 13 seconds and height of 42 feet. This is the deep water wave criteria where the water depth is at a sufficient depth where the wave does not interact with the sea bottom and transform and lose energy as it approaches the pipeline area and shoreline. The wave length associated with the 13 second wave is 865.28 ft. The water depth where this deep water wave begins to feel sea bottom effects and starts its energy transition loss toward the pipeline and shoreline is 432.64 feet. To determine the waves' effects and forces, the pipeline in the sea bottom will experience hydrographic surveys of the sea bottom contours from the proposed pipeline shoreline location along a line that is perpendicular to the offshore contour lines to the 432.64 ft transition water depth is needed. As the design wave moves from the deep water transition depth toward the shoreline it encounters frictional resistance in its interaction with the sea bottom. During this transition wave energy is lost, wave length shortens, wave velocity deceases, wave height decreases and the period stays constant. These changes are shown in Figure 1a-d.

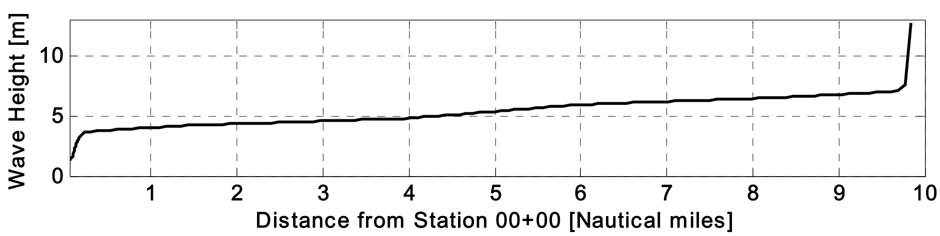

Figure 1a Offshore evolution

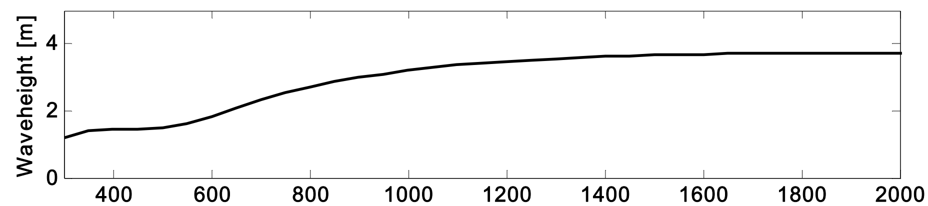

Figure 1b Nearshore evolution of wave height

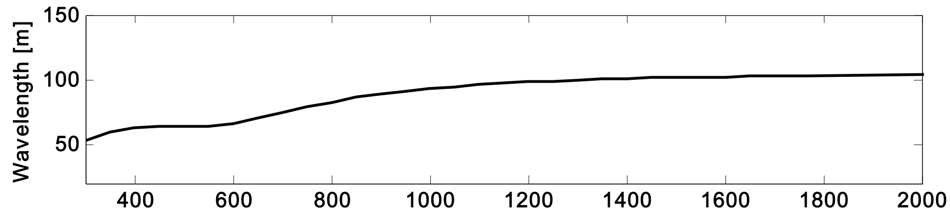

Figure 1c wavelength

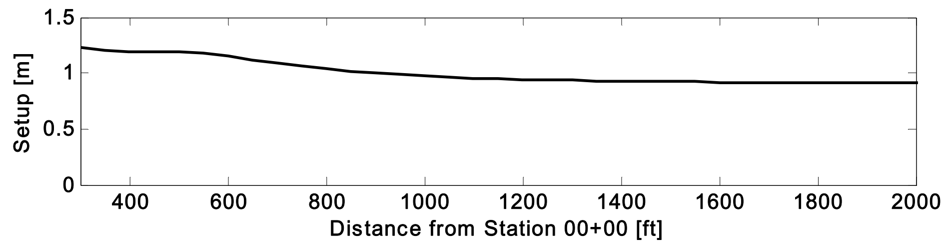

Figure 1d Setup as a function of offshore distance measured from station 0+00

Table 2 Maximum Speed and Dynamic Pressure at the Mud-Line Level During Design Conditions

		Pressure	
Station	**Speed**	**Dynamic**	**Pore**
X	**U_0**	**p_0**	**p_b**
[ft]	[m/s]	[N/m2]	[N/m2]
200	0.77	4021	3515
250	0.92	4835	4364
300	1.03	5860	5381
350	1.01	6603	6150
400	0.98	6857	6437
450	0.96	6859	6453
500	1.02	7036	6616
550	1.13	7657	7203
600	1.20	8582	8103
650	1.31	9696	9203
700	1.36	10727	10239
750	1.40	11608	11131
800	1.42	12364	11898
850	1.44	12970	12515

		Pressure	
Station	**Speed**	**Dynamic**	**Pore**
X	**U_0**	**p_0**	**p_b**
[ft]	[m/s]	[N/m2]	[N/m2]
950	1.45	13837	13398
1000	1.47	14207	13770
1050	1.49	14567	14133
1100	1.48	14811	14383
1200	1.49	15205	14784
1250	1.50	15356	14937
1300	1.51	15533	15114
1350	1.51	15665	15247
1400	1.50	15752	15338
1500	1.51	15828	15416
1550	1.50	15876	15465
1600	1.51	15951	15540
1650	1.51	15988	15579
1700	1.51	16010	15602

This transition at any location on the route of the wave, as it approaches the shoreline, produces corresponding forces on the sea bottom below by the Airy theory and Fenton's theory. During the transition of the waves, the stability of the wave needs to be determined. As the forward wave velocity decreases, due to bottom drag, the peak of the wave can become unstable and break. Two types of breakers occur, a spilling breaker or a plunging breaker. These breaking waves exert increased sea bottom pressure than a decaying wave. The results of these forces are in Table 2.

The next step to determine the stability of the buried pipeline against pull out from wave force dynamics is to assume a burial depth, submerged weight of the pipe and backfill materials above the pipe. Significant offshore soil borings along the pipeline route need to be taken to determine if the existing material is sufficient to support the pipe and/or can be used for any backfilling. If sea bottom scour forces are sufficient enough that natural sea bed materials can be displaced, then a combination of native/imported backfill materials and armor stone is assumed. Now a pull out analysis can be completed to see if the submerged weight of backfill and pipe, with water in it, is greater than the hydrodynamic pull out forces for the waves. These forces and the factor-of-safety are shown in Table 3.

Since the FS is greater than 2 at all locations along the pipeline route, the pipe material and backfill material assumptions are good. If the FS was not acceptable, then pipe wall thickness, burial depth, thickness of armor stone layer or higher specific gravity of stone would have to be reevaluated. In areas subject to seismic activity these forces need to be compared with the wave dynamic forces with the largest controlling the design criteria. The possibility of a seismic event and 50 year storm happening at the same time is extremely remote. Since the specific weight of the pipe with water in it is less than the surrounding backfill material and liquefaction of this backfill material is possible the pipe becomes positive buoyant relative to the surrounding backfill. Ishihari and Yamazaki (1984) state "liquefaction will occur if the cyclic strength of the soil is less than the wave induced cyclic stress ratio of the soil". The results of the liquefaction analysis, along the pipeline route, is shown in Tables 4, 5 and 6. The liquefaction results show that the assumed buried pipe depth is lower than the liquefaction depth at all locations along the pipeline route. However, a future potential problem exists that must be accounted for. The backfill material assumed, usually #57 stone, is free draining. If a fine material, such as sand, is used as backfill above the free draining stone over a period of time, and significant wave cycles, the fine material can migrate down into the free draining layer making it non free draining and susceptible to liquefaction. An easy solution to prevent the migration of fines into the free draining material it to encase the free draining material in a geotextile fabric.

Table 3. Stability of Pipeline Against Uplift Forces

Station (ft)	Water Depth, d (ft)	Wave Length, L (m)	Seabed Dynamic Pressure, p_0 (kN/m^2)	Factor of Safety = F_{down}/F_{up}
1+00	0.52	24.5	2.80	2.25
2+00	1.68	41.1	4.02	2.25
3+00	2.38	53.3	5.86	2.21
4+00	3.51	63.0	5.86	2.21
5+00	3.51	63.9	7.04	2.20
6+00	3.78	66.4	8.58	2.79
7+00	4.75	74.5	10.73	2.75
8+00	5.88	82.8	12.36	2.73
9+00	6.83	88.9	13.45	2.72
10+00	7.44	92.8	14.21	2.18
11+00	8.02	95.9	14.81	2.18
12+00	8.38	98.1	15.21	2.18

Table 4 Prediction of the Depth of Liquefaction of Soil Below Seabed

Station (ft)	Water Depth, d (m)	Wave Length, L (m)	Design Wave Height, H (m)	Depth of Liquefaction, z (ft)
1+00	0.52	24.5	0.7	N/A
2+00	1.68	41.1	1.9	N/A
3+00	2.38	53.3	2.5	N/A
4+00	3.51	63.0	3.2	N/A
5+00	3.51	63.9	2.6	N/A
6+00	3.78	66.4	3.5	0.03
7+00	4.75	74.5	4.9	8.10
8+00	5.88	82.8	5.8	10.94
9+00	6.83	88.9	6.3	11.56
10+00	7.44	92.8	6.5	11.04
11+00	8.02	95.9	7.0	12.96
12+00	8.38	98.1	7.0	11.80

Table 5. Trench Depth from Seabed – 84" Internal Diameter Pipe

Station (ft)	Cover Layer (ft)	Filter Layer (ft)	Under Layer (ft)	Bedding Layer (ft)	Total Depth (ft)
0+10 to 6+00	3.15	1.00	0.50	3.50	16.48
6+00 to 9+50	5.70	1.45	0.50	3.50	19.48
9+50 to 12+00	3.55	1.00	0.50	5.50	18.88

If the offshore termination or the pipeline is a nozzle the design and protection of same is an independent design from the buried pipeline. It will also be subject to the same vertical hydrodynamic forces of the pipeline but also the oscillating horizontal forces and scour for the portion above the sea floor. When evaluating the most suited pipe material for the project, all of these forces must be considered.

Table 6. Factor of Safety

Station (ft)	Depth of Liquefaction, z (ft)	Trench Depth (ft)	Factor of Safety
1+50	N/A	16.48	N/A
4+50	N/A	16.48	N/A
5+00	N/A	16.48	N/A
6+00	0.03	19.48	N/A
7+00	8.10	19.48	2.4
8+00	10.94	19.48	1.8
9+00	11.56	19.48	1.7
10+00	11.04	18.88	1.7
11+00	12.96	18.88	1.5
12+00	11.80	18.88	1.6

The initial cost of the pipe should not be the determining factor for pipe material selection. The installed cost of the pipeline must be considered. A pipe with a high submerged weight can significantly reduce the amount of imported backfill and armor stone to resist pipe pull out and/or flotation of the pipeline. To date Prestressed concrete cylinder pipe (PCCP) has been the most economical selection, especially in diameters of 36" and greater. Prestressed concrete cylinder pipe (AWWA C301) is commonly manufactured with twenty foot lay lengths and steel end rings, providing critical design elements such as reduced number of joints, heavier lay lengths to offset pullout forces and external testable joints.

Offshore intakes/outfalls due to their location and relative unpredictable environment may make repairs difficult, time consuming, extremely costly and completely cripple any operation that is dependent on them due to insufficient design considerations.

CONCLUSIONS

You need to get the design right the first time. As the British say “The end product is the direct result of the effort put into it.”

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Fast Track Relief to Midland's Emergency Thirst

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Abstract

Using the traditional design-bid-build (DBB) project delivery system, the T-Bar Ranch Well Field Development & Delivery System project would likely have taken upwards of three or four years to complete. With the City of Midland, Texas, deep in a severe drought, an alternative delivery system was a necessity. The City was facing the probability of being cut off from their primary source of water in less than fifteen (15) months. By means of the design-build (DB) delivery system – in this case, design-build-finance-operate (DBFO) – the Project Team took tasks that, using the DBB system would have been completed in a linear manner, and overlapped them, tackling them in conjunction with one another, significantly decreasing the overall project schedule and ultimately the cost to build. Diligent management of the land acquisition, design, material manufacturing, and construction resources delivered this project ahead of schedule and under budget.

Keywords: Design build; Water pipeline; Alternative delivery method.

INTRODUCTION

In the early 1960s, the Midland City Council purchased 8,903 hectares (22,000 acres) of land and water rights on the T-Bar Ranch located in Winkler and Loving Counties, Texas. These water rights included 678,415,000 m^3 (550,000 acre-feet) of water in the Pecos Valley Alluvium Aquifer. The project to deliver that water to users was one of the largest waterline projects in the United Sates, as well as one of the area's largest public works projects.

OIL AND GAS, YES – WATER, NOT SO MUCH

As many communities still struggle to climb up from the economic downturn, Midland is booming with a healthy gas and oil industry that has added thousands of jobs over the past several years. That industry – and the workers it attracts – not only requires water, but also increases demand for that precious resource.

According to the United States Bureau of Labor Statistics, the unemployment rate in Midland was only 3.4 percent in December of 2012 (United States Department of Labor Bureau of Labor Statistics 2015). The city was experiencing dramatic economic growth, demonstrated with spectacular consumer activity, increased airline boardings and auto sales, and rapid hotel construction to meet increased demand. Most would consider chambers of commerce that expect double-digit growth to be incredibly fortunate.

Of course, there’s also a downside. Because of the rapid growth, people struggled to find a place to live, others suffered in traffic like they have never seen before, and some businesses were desperately seeking workers to help them keep up with customers. People were having to book hotels months in advance due to the demand for rooms. These economic drivers themselves created another impact to the city: the demand for water.

Since 2010, the area around Midland has suffered extreme to severe drought. According to the National Weather Service, in 2011, the area had only 13.89 cm (5.47 inches) of precipitation as compared to its annual average of 37.08 cm (14.6 inches) per year, the third worst in recorded history (National Weather Service 2015). The Colorado River Municipal Water District, which supplies the majority of Midland’s water, depends on three surface reservoirs. Two of them were dry and the third was at twelve (12) percent capacity with a prediction that the water supply would run out by June 2013. This lack of rainfall led to water restrictions and implementation of drought contingency plans across Texas.

The city issued a Request for Proposals (RFP) with the prescriptions to finance, design, construct, and operate a water conveyance system that would deliver a maximum of 75,708 m^3 (20 million gallons a day) to the city by the end of May 2012 and have the capacity to be expanded to a maximum of 132,489 m^3 (35 million gallons a day) by 2029 (refer to Figure 1).

The City of Midland (City) owns 26 sections of land in Winkler County, Texas and 7 sections of land in Loving County, Texas collectively known as the T-Bar Ranch Well Field, approximately 70 miles northwest of the City. The City is seeking proposals for the development of this water resource and delivery of the water to the City by 2013 in the following amounts:

2013-2029	*Maximum of 10 million gallons per annual average day*
	Maximum of 20 million gallon per maximum day
2029-contract duration	*Maximum of 28 million gallons per annual average day*
	Maximum of 35 million gallons per maximum day

Figure 1. Excerpt from RFP

PROJECT COMPONENTS

- 44 public water supply wells spaced on average half-mile apart on the T-Bar Ranch property
- 32 km (20 miles) of new well field all-weather access roads
- 7,570m^3 (2 million gallon) well field storage tank
- 75,708 cubic meter (20 million gallon) per day high-service pumping station
- 18,927m^3 (5 million gallon) intermediate storage tank
- Intermediate chlorination facility
- Terminal facility with pressure control valve
- New electrical power feeds to all facilities
- Telecommunication towers along route to handle SCADA communication

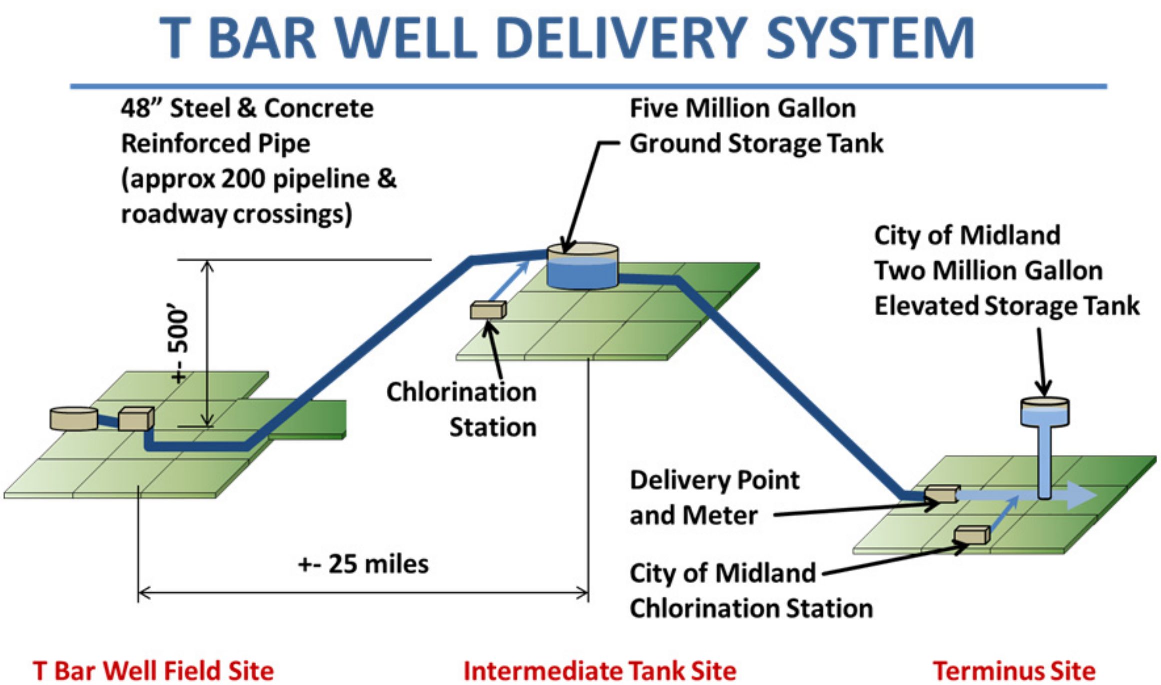

Figure 2. System Layout

THE PROJECT

The design of the well field and pump station was relatively easy for the simple fact that the required land was already in hand. But without a route selected, the design of the 58-mile transmission main and intermediate storage tank presented a challenge.

Lying between the T-Bar Ranch well field and the City of Midland are roughly 96 km (60 miles) of obstacles, including sand dunes, prairie, pasture, three large and active oil fields, numerous cattle ranches, one wind farm, several hundred pipelines, and more than 55 landowners who take great pride in owning their land.

Route Selection & Land Acquisition

The Midland County Fresh Water Supply District Number 1 (MCFWSD1), possessing the power of eminent domain, could provide for the shortest route – a straight line from A to B (refer to Figure 3).

Figure 3. Original Pipeline Route

However, the decision was made early on that condemnation would only be used as a last resort, and even then, only in extreme cases. More than once, the Project Team made what amounted to a major route change to avoid having to exercise that authority which not only avoided confrontation, but also allowed the project to remain on schedule.

The dense oil field maze of pipelines also drove a number of the alignment reroute decisions (refer to Figure 4).

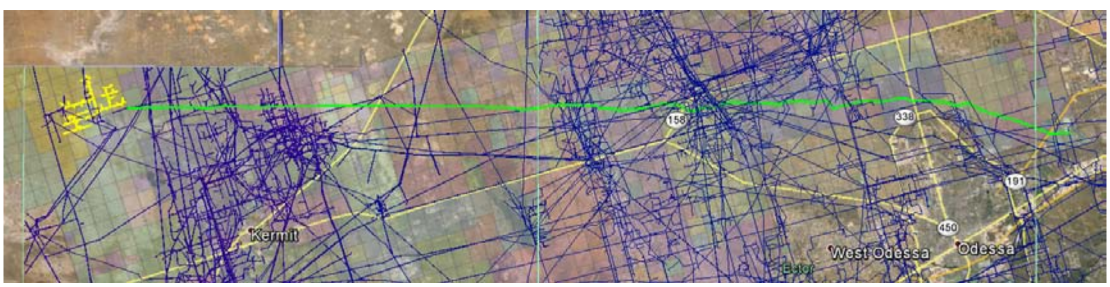

Figure 4. Maze of Oil Field Pipelines and Land Owners Color Coded by Parcel

The decisions to route around the permanent oil field infrastructure and immoveable sand dunes was cut and dry; the decision to reroute around potentially difficult land owners was not as cut and dry and a heavy responsibility was placed on the shoulders of the land acquisition team to make educated and accurate decisions of what properties to route through and around (refer to Figure 5).

Figure 5. Rerouted Pipeline Alignment Versus Original Alignment

With the final route selected, the acquisition of land, well underway, was now critical to the on-time completion of the project (refer to Figure 6).

Figure 6. Finalized Pipeline Route

Managing the Land Acquisition, Design and Construction

After spending a couple of months determining the final pipeline route, the Project Team now had the topographic data needed to move forward. Acquiring the 93 km (58 miles) of easements, designing the pipelines, and manufacturing pipe needed to be phased and managed diligently.

Splitting the transmission main into four segments and selecting the lay direction of each – east to west, or west to east – gave the design team, land acquisition team, and pipe manufacturers the information essential to completing their tasks.

Land Acquisition

Because the land acquisition activities were already in progress, the decisions of where to split the four segments and which direction the pipe would be installed were influenced by the landowners who were the most likely to or had already signed an easement agreement. A schedule was developed and a level of priority was placed on each piece of land giving clear direction for the land acquisition team. With the clear objective to not exercise our right to condemn, the land acquisition team was diligent in educating, communicating, and working with the landowners to provide a win/win situation for everyone. In less than six months of the Notice to Proceed, all agreements were in hand, none of them through condemnation.

Pipeline Design & Pipe Manufacturing

Prior to the final route selection, preliminary design of the pipeline was underway and three pipe manufacturing companies were weeks into producing pipe. With a general alignment of the pipeline being fairly certain early on, the decision was made to release certain quantities of numerous pressure classes and types of pipe without final design.

Dividing the design of the pipeline into four segments of approximately 24 km (15 miles) each undoubtedly took some time and had the potential to stop the forward movement of the project. The answer was to split each of the four segments into three sub-segments (with the exception of Segment #4 which was divided into two sub-segments) – a total of 11 each 8 km (5 mile) segments to design. This allowed design packages to be prepared and released early to the pipe manufacturers in order to expedite pipe production.

To achieve timely production of pipe, three different pipe manufacturers were utilized at four different production plants in the United States. Pipe materials included the following:

- Polyurethane coated steel pipe (AWWA C200); both lap welded joints and gasketed joints up to CL225
- Cement mortar coated steel pipe (AWWA C200)
- Bar-wrapped concrete steel cylinder pipe (AWWA C303)

Polyurethane coated steel pipe (all welded joints) was used for the higher pressure portion of the system beginning at the high service pump station and continuing approximately three-quarters of the way to the intermediate tank. Bar-wrapped concrete cylinder pipe was used in the lower and medium pressure segments – 8 km (5 miles) upstream of the intermediate tank and approximately 16 km (10 miles) on the downstream side. This decision was more of an economics-based decision than an engineering-based decision due to the pipe diameter-to-wall thickness ratio, or D/t ratio, controlling steel pipe design (equivalent minimum pressure class of CL200 for polyurethane coated steel pipe). Cement mortar coated steel pipe was used in the 150 psi to 175 psi working pressure range downstream of the bar-wrapped concrete cylinder pipe segments. A fourth pipe manufacturer was needed for production purposes and this segment was ideal for cemented mortar coated pipe since wall thickness was based on a design yield stress of 18 KSI (per AWWA M11) as well as the established D/t ratio, which allowed for the pipe to be designed in accordance with actual system working pressures. Conversely, polyurethane coated steel pipe would have been over-designed for the same working pressures based on the D/t requirement. Polyurethane coated steel pipe (gasketed joints) was used for the last section (Segment 4), beginning approximately 20 km (12.5 miles) upstream of the terminus facility (McCure, 2014).

A summary of the pipe material and pressure classes are as follows:

- **Segment 1A, 1B, 1C:** AWWA C200 Polyurethane Coated Steel Pipe (lap welded joints) – CL200, CL225, CL250, and CL275.
- **Segment 2A:** AWWA C200 Polyurethane Coated Steel Pipe with welded joints – CL175 and CL200.
- **Segment 2B, 2C:** AWWA C303 Bar-wrapped Concrete Cylinder Pipe with gasketed joints – CL150.
- **Segment 3A:** AWWA C303 Bar-wrapped Concrete Cylinder Pipe with gasketed joints – CL150.
- **Segment 3B and 3C:** AWWA C200 Mortar Coated Steel Pipe with gasketed joints – CL150 and CL175.
- **Segment 4A and 4B:** AWWA C200 Polyurethane Coated Steel Pipe with gasketed joints – CL200 and CL225.

Managing Resources

The installation and commissioning of 93 km (58 miles) of 121.9 cm (48-inch) pipeline in under 10 months would require more resources than just four pipe crews. In order to obtain the production needed out of these four crews, it was vital that the preconstruction activities stay in front of them and the post-installation activities stay close behind.

The use of conventional hydraulic excavators was not practical for trenching through the hard caliche / limestone material along the eastern portion of the alignment; therefore, pre-trenching was performed using large chain-saw type trenchers. The trenching machines served a dual purpose – cutting the ditch for faster installation and sufficiently grounding up and pulverizing the hard native caliche / limestone soils which were stockpiled and later segregated by running through a high capacity screener and eliminating the need for imported bedding.

Two of eleven of the world's largest trenchers – Trencor 1860 (Figure 7) – were mobilized on this project along with numerous smaller machines.

Figure 7. Trencor 1860 Rock Trencher

These giant trenchers, weighing nearly 226,796 kg (500,000 lbs.) each, limited how close they could operate to the hundreds of petroleum pipelines that the project traversed. The skips the trencher left behind were often 18 to 24 km (60 to 80 ft.) wide which needed to be excavated prior to the mainline crew's arrival. Smaller crews with hydraulically operated rock hammers on excavators were employed to accomplish this. Tunneling, land clearing, and fencing crews also preceded the installation crews.

The primary focus of the four installation crews was to install pipe with minimum slow-downs. Following closely behind them were four support crews to build out the appurtenances, including blow-offs valves, air release valve structures, and cathodic protection, and restore the right-of way.

In total, more than 130 craftsmen were working on the transmission main at the peak of construction. The challenge this presented was finding quality workmen in the surrounding market of Midland, Texas. The hub of the Permian Basin oil boom had an unemployment rate around three percent and forced the Project Team to bring in nearly all craftsmen from outside the area, mobilizing crews from Wyoming to Florida.

CONCLUSION

Although the design-build project delivery method is relatively new for linear construction projects in the water industry, the successful completion of T-Bar Ranch Well Field Development & Delivery Project demonstrates how effective it can be when properly managed. Having the ability to overlap tasks shortened the overall project duration by multiples (refer to Figure 8, Project Schedule). Setting bold but realistic deadlines from the outset while also having flexibility in "how things get done" but remaining rigid on "when things get done" was critical to finishing on time.

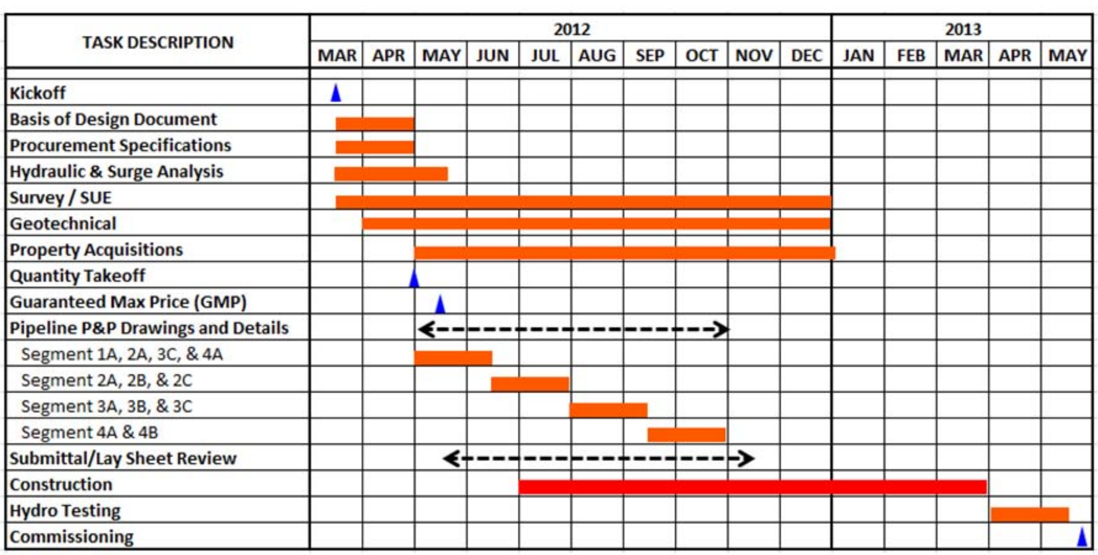

Figure 8. Project Schedule (McCure, 2014).

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Share the Road: Challenges and Opportunities Facing Joint Pipeline and Roadway Construction Contracts

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Abstract

Dallas Water Utilities (DWU) supplies treated water to 2.3M people in the Dallas-Fort Worth Metropolitan area (DFW). The East Side Water Treatment Plant (ESWTP) is a 440-MGD plant that has the capacity to meet nearly half the City of Dallas' customer demands including its wholesale customer cities. A regional large diameter pipeline project called the Southwest 120/96-inch Water Transmission Pipeline Project (Southwest Pipeline) will provide redundancy and increase service capacity to meet the growth of current and future DWU customers in Southern Dallas County. The Southwest Pipeline will serve multiple jurisdictions over its 32 miles of urban area. During the planning stages of the project, multiple roadway expansion projects were identified along the pipeline corridor that would overlap with the pipeline project and involve coordination with multiple jurisdictions and agencies. In many cases, joint construction contracts were deemed the best delivery method for both the roadway and the pipeline projects. The coordination between the multiple agencies for joint construction contracts presents both challenges and opportunities. This paper will discuss the challenges and opportunities facing joint pipeline and roadway construction projects and will cover lessons learned during several phases of the project, including planning, design, bidding, and construction.

I. INTRODUCTION

The coordination among multiple public agencies for joint construction contracts presents both challenges and opportunities. There are two segments of the project that are currently in a joint construction contract with a roadway project that will be discussed; one located on Bonnie View Road and one located on Telephone Road. DWU is the agency responsible for the Southwest Pipeline project while City of Dallas Public Works Department (Telephone Road) and Dallas County Public Works Department (Bonnie View Road) are the participating entities for the roadway improvements.

This paper will discuss the challenges and opportunities facing joint pipeline and roadway construction projects and will cover lessons learned during several phases of the Southwest Pipeline project, including planning, design, bidding, and construction. There are generally four main phases of the joint construction contract that will be discussed in the following sections.

The first phase of the joint construction contract is planning, which involves several components. Right-of-Way (ROW) acquisition, overall project schedule, and funding are some of the major topics that should be covered in this phase of a project. Coordination on these items early on in the process will determine whether a joint construction contract is the right delivery method for both projects. The next stage of the joint construction contract is design. Construction sequencing, traffic control, utility and alignment coordination, and packaging the contract documents for bid are key components that should be coordinated throughout all stages of the design phase. Prior to the bidding phase, the construction schedule and bidders qualifications must be coordinated by the public agencies. There may be specific contractor qualifications required for each of the roadway and the pipeline projects which may impact the bidding and award phase. After the project has been awarded, the construction phase will begin.

This paper will discuss major topics that must be coordinated during the construction phase such as construction scheduling, submittals review, quality control, material delivery, and inspection services. Despite the challenges associated with joint construction contracts, they often offer opportunities to save time, money and reduce public impact for all parties associated with the project.

II. PLANNING

In the initial planning stages of a large diameter pipeline project especially in urban areas, it is important to coordinate with local agencies and jurisdictions to determine if there are any roadway expansion projects along the proposed pipeline corridor. If roadway expansion projects are identified, a joint construction contract could be a potential project delivery method that may benefit all parties. During the planning stages of the Southwest Pipeline project, multiple roadway expansion projects were identified along the potential locations of the pipeline corridor. Identifying the future roadway projects along the pipeline corridor was part of the reasoning behind selecting the current pipeline corridor. The roadway projects overlapped with the pipeline project which involved coordination with multiple jurisdictions and agencies along the 32 miles of pipeline. In many cases, joint construction contracts were deemed the best delivery method for constructing the roadway and the pipeline.

A. RIGHT-OF-WAY ACQUISITION

A major part of the planning phase for both projects is ROW acquisition. Typically a roadway expansion and large diameter pipeline projects require a ROW acquisition. In the initial planning phase, the number of parcels affected by both projects need to be determined. This step may occur independently by both projects and then any overlap in ROW can be determined. Another important step is to determine the type of ROW acquisition that is needed for both projects, whether it is fee simple or an easement, either temporary or permanent. A fee simple acquisition would mean full ownership and rights to the property, whereas an easement can either be acquired for construction (temporary) or future maintenance (permanent) but is not a full ownership of the property. Typically, roadway ROW is acquired by dedication or fee simple acquisition. For pipeline projects, the ROW is typically acquired by easement. For Southwest Pipeline, the majority of the ROW is acquired by fee-simple acquisition. In a joint construction contract, the ROW is typically acquired by agency leading the roadway expansion project. When the roadway is expanded the additional ROW acquired for the expansion becomes public ROW. This public ROW can be used for the installation and maintenance of the large diameter pipeline project. Without the roadway expansion project, the ROW would need to be acquired by the pipeline agency. Typically, the agency leading the pipeline project will contribute financially to the ROW acquisition led by the roadway agency when mutual benefit is achieved. Utilizing the ROW acquired by the roadway project may also allow the pipeline flexibility in terms of construction sequencing, which will be discussed in a later section.

Although the roadway expansion project will determine the extent of proposed ROW and eventually acquire the additional land, the pipeline project may need to consider the possibility of additional ROW. There must be adequate space for pipeline construction and future maintenance, but there also may be new water, sewer, and drainage utilities that will be installed or relocated as part of the roadway expansion project. It is important to consider these possibilities when determining the required ROW for the pipeline. In many instances, the entity leading the pipeline project may consider additional permanent or temporary construction easement or fee simple acquisition to supplement the ROW acquired by the roadway project.

It was determined in the planning phase of Southwest Pipeline that for the most part, the proposed ROW acquisition for the roadway expansion project would be adequate for the installation and maintenance of the pipeline. However, additional ROW would need to be acquired in all other areas along the pipeline corridor that did not involve a roadway expansion project.

The Bonnie View segment of Southwest Pipeline already had adequate public ROW construction of the pipeline and expansion of the road so there was no additional ROW was acquired. The Telephone Road segment, on the other hand, required additional ROW along the entire length of roadway. DWU started the ROW acquisition much earlier than the roadway agency, so in this case, the pipeline agency lead the ROW acquisition in fee simple acquisition and the roadway agency utilized the ROW acquisition for the roadway.

B. OVERALL PROJECT SCHEDULE

The overall project schedule for both the roadway and pipeline should be considered in the early planning stages. Both projects may have different drivers and may result in different project timelines. Roadway projects typically are driven by economic development. Pipeline projects are typically driven by future water demands and redundancy. The pipeline agency must decide if the roadway design and construction schedules are adequate to allow for a joint construction contract. If this is not an option, an alternate route or additional ROW outside the roadway project ROW may be considered to avoid constructing the pipeline along the proposed roadway route after it has been constructed. The roadway entity may also be flexible in their schedule to allow for the joint construction contract to occur. The Southwest Pipeline project entities were not heavily involved in the roadway construction schedule since the construction of the two segments of pipeline was not urgent. Ongoing project coordination meetings between both agencies during the design phase are critical to discuss and resolve issues with alignment, utility conflicts, depth of cover, and the overall project schedule.

The roadway expansion projects identified along Southwest Pipeline alignment corridor were merely in the initial planning stages at the time of discovery. This allowed the Southwest Pipeline project ample time for design and coordination with each roadway entity along the pipeline alignment corridor.

C. FUNDING

A major part of the planning phase is determining the required funding for both projects. As separate projects, ROW acquisition, design, and construction would be funded separately. In a joint construction contract, both agencies will be contributing to fund their portion of the project. Funding must be decided during the planning phase to avoid any issues that may arise in the future during design and construction of the projects. This project required executing a Project Specific Agreement (PSA) between the two agencies and will require approvals from both the pipeline and roadway agencies.

There are two segments of the Southwest Pipeline project that have been designed and have been or are currently in the construction phase. The first segment of this pipeline project is 3000 linear foot (LF) in length and runs along Bonnie View Road in Dallas, Texas. This roadway is being expanded to a four lane divided roadway. In this case, the pipeline was funded entirely by Dallas Water Utilities, while the roadway was co-funded by Dallas County and City of Dallas. The second segment is about a mile long and runs along Telephone Road also in Dallas, TX, which is also being expanded into a four lane divided roadway by the City of Dallas Public Works Department. Although both Dallas Water Utilities and the Public Works Department are part of the same overall entity, the City of Dallas, both projects were funded separately throughout design, bidding and construction phases. Without ongoing coordination with both of these parties, the joint construction contracts on both segments would not have run as smoothly.

III. DESIGN

There are several items to coordinate during design. The alignment of both projects, traffic control required for construction, future and existing utilities, construction sequencing, and the development of contract document are major items that need to be coordinated between all parties involved.

A. ALIGNMENT & DEPTH OF COVER

In a joint construction contract with separate entities and designers, the initial roadway alignment and pipeline alignment will most likely be determined independently. During the design phase of the project is when the final alignment should be coordinated for each. The roadway project should provide proposed typical sections along the roadway which will allow the pipeline design team to develop a final alignment that will not affect the proposed roadway and utilities improvements. Special attention to the minimum depth of the pipeline will be a critical factor to accommodate utility crossings especially storm sewer systems. If the pipeline will be constructed first, then the depth of cover should accommodate any earth cuts along the pipeline alignment to construct the new road.

B. UTILITY COORDINATION

Utility coordination is a major part of the design process of any major roadway or pipeline project. The roadway project may have proposed water, sewer, and drainage improvements. Furthermore, existing utilities along the existing roadway may need to be relocated to resolve conflicts with the proposed roadway and pipeline improvements. It may also be possible that these improvements are designed and funded by a separate entity, developer, or another department of the same entity. Coordination early on in the design must include provisions for future utility improvements and at the very least coordination with existing utility owners.

Southwest Pipeline required very extensive utility coordination for existing utilities along the existing roadways in other urban areas, but very little along Bonnie View Road and Telephone Road. However, both roadway projects included drainage improvements in their design which affected the pipeline project. The roadways were lowered and underground drainage facilities like culverts and storm drains were installed, all of which had to be coordinated with the pipeline design. With new or expanded roadways, the requirements for drainage design become more involved. Culvert crossings required to direct runoff across the road may be very large and storm drainage piping may run along the entire length of the roadway to direct runoff to those culverts. Drainage improvements may require the vertical profile of the pipeline to be lowered under the roadway to avoid those culverts, storm drains and inlets along the road. As highlighted above, depth of cover is critical for the utility coordination aspect of design. Both segments of Southwest Pipeline required significant drainage improvements that changed over time during the development of the roadway design. Sometimes that means changing the alignment or profile of the pipeline to allow for adequate placement of these drainage improvements above and across the proposed pipeline.

There may also be water and sewer line improvements along the roadway which require ongoing coordination as well. Bonnie View Road required improvements to both water and sewer which had to be coordinated significantly with the drainage, roadway, and pipeline design. The design teams of all the different utility projects should coordinate together during design to avoid potential construction and phasing issues.

C. CONSTRUCTION SEQUENCING & TRAFFIC CONTROL

During the final design phase, construction sequencing must to be coordinated between both parties involved in the roadway and pipeline projects. There can be multiple options for how the roadway and pipeline are constructed, but if it is thought out far in advance of construction it will allow for a smoother running construction phase. The roadway project will most likely design the traffic control that will be used during construction, but there may also be issues that arise with constructing the pipeline simultaneously.

For the Southwest Pipeline project, the design of both projects traffic control measures were well coordinated and thought out completely along the entire length of the project. Considerations for traffic control also include maintaining access to nearby houses or buildings in the area, as well as providing two-way traffic along the roadway at all times during construction. Temporary pavement may be required along the roadway in tighter areas with less ROW or possibly at intersections, which was the case in the Southwest Pipeline project. Figure 1 below shows the construction area along Bonnie View Road. The south bound lanes are closed off while two-way traffic is being maintained along the recently constructed north bound lanes. The design of Bonnie View Road allowed for the two north-bound lanes of the roadway to be constructed while maintaining traffic control on the existing two lane road. Two-way traffic was then moved over to the new north-bound lanes while construction of the pipeline could then occur. During construction of the pipeline along Bonnie View Road, roadway construction was still occurring south of where the pipeline would be installed. This allowed for continual construction of the roadway project while the pipeline was being installed.

Figure 1. Bonnie View Road Traffic Control

After construction of the pipeline, the roadway project would finish up the paving and drainage improvements along the pipeline corridor. Figure 2 below shows the construction of the 96-inch pipeline and pipe trench in the future north bound lanes of Bonnie View Road. The south bound lanes have been completed and are shown to the left of the safety fence.

Figure 2. Bonnie View Road Construction Sequencing

D. DEVELOPMENT OF CONTRACT DOCUMENTS

During design of both the roadway and pipeline projects, contract documents will need to be prepared for both projects. The contract documents should include any provisions for the contractor required during construction. There may be different constraints for each project separately, which shows the important of coordination early on during design. There may also be different requirements for the contractor for each project that the contractor would need to be aware of during bidding. One agency has to take the lead during bidding and construction. This needs to be determined early on in the design phase. Typically the agency with the higher construction estimate or the one with more complex project will take the lead. The roadway project entity was the lead agency in both segments of the Southwest Pipeline project. Typically the front end documents of the lead agency are used for the joint project contract document. The roadway project entity was the lead agency in both segments of the Southwest Pipeline project. The following should be considered when doing so:

1. **Bid Items:** Some bid items may be common for both the roadway and pipeline projects. Items such as bonding, mobilization, Storm Water Pollution Prevention Plan (SWPPP), trench safety, traffic control, concrete barriers, to name a few, need to be coordinated to avoid duplication. The roadway entity on both segments of the Southwest Pipeline project covered the costs associated with the SWPPP and traffic control, but the pipeline entity was responsible for the costs of trench safety and concrete barriers associated with the pipeline portion of the project.
2. **Bidders Qualifications:** It is very important to develop minimum bidder qualifications prior to the bidding of the joint construction project. A roadway contractor may not be qualified to install a large diameter pipeline, and the reverse is true. Typically, the construction cost estimate for the roadway and the pipeline will determine the leading general contractor. Bidder Qualifications were clearly stated for the both roadway and pipeline projects for Southwest Pipeline by establishing the desired minimum previous experience for the pipeline and roadway separately. The project qualification section accounted for the fact that either the roadway or the pipeline contractor may be subcontractor to a general contractor. Qualifications for the pipeline contractor were established based on years of experience of installing large diameter steel pipe as well as length of pipe installed and resumes of key personnel on the project.
3. **Terms and Conditions (T&Cs):** It is very common that both agencies use different T&Cs. Typically the leading agency will use their own T&Cs for the joint contract. In some cases, a hybrid T&Cs will be used to accommodate the needs of both agencies. It is important for both agencies to review and coordinate the joint project T&Cs prior to bid to avoid any conflicts or confusion. This may include but not limited to liquated damages, holidays, change order approval process, payment approval process, incentives, substantial completion and closure requirements, to name a few. T&Cs from the leading roadway agency were used on the Southwest Pipeline project.
4. **Payment Terms:** The typical payment terms used by both agencies may be different and need to be harmonized before the bid phase. Potential conflict may create confusion during construction and may cause project delays. The pipeline entity provided separate payment terms for the Southwest Pipeline project that were specific for the pipeline related materials and installation requirements.

5. **Quality Control (QC):** Quality control requirements for each scope of work should be clearly defined in the contract document. It is not uncommon to see different QC measures and requirements for each project as long as it is clearly defined in the contract document to avoid any confusion during construction. A roadway project may focus on concrete compression tests or soil compaction testing, while a pipeline project may focus more on embedment and soil compaction, holiday testing, and welding procedures, especially for steel pipeline. A common conflict that has been seen in the Southwest Pipeline project is the frequency and requirements of density testing. The roadway and pipeline agencies required different levels of quality control requirements for certain tests. Although the roadway agency provided the quality control on this project, the pipeline entity provided separate quality control personnel for the specific requirements related to the pipeline installation. It is recommended that these quality control measures be well defined in the contract documents and be made clear to the contractor during bid phase and be reminded again prior to the start construction phase.
6. **Technical Specifications:** Roadway and pipeline constructions are governed by different technical standards. This is common as long as it is clearly stated in the contract documents which technical standards are governing the construction of each project. In case of a conflict between the two projects, a hierarchy of standards should be established to avoid any unforeseen conflicts. It is very common to see the drawings and technical specifications for both projects signed and sealed separately in different volumes as long as any conflicts are resolved and clear direction are established in the front end documents of the joint projects. Where same material are required for both projects (such as small diameter PVC pipe or valves), the approved manufacturer list should be coordinated. This may be an issue if both agencies have different approved manufacturers list or requirements. Technical specifications were developed for the Southwest Pipeline project that were in addition to locally governed standards and the roadway project specifications to ensure all aspects of the large diameter pipeline construction were covered.

IV. BIDDING

A. CONSTRUCTION SCHEDULE

A joint construction contract requires a conformed construction schedule that is agreeable by both the roadway and pipeline agencies. Coordination and estimation of the time required to complete construction of both projects is required prior to bidding of the project. The construction schedule will depend on the construction sequencing that was developed during the design phase of the project.

B. SUCCESSFUL BIDDERS

The two segments of the Southwest Pipeline project have both been bid, awarded, and are currently in construction. Both projects were bid with specific pipeline installation qualifications and for both projects, the successful bidders were pipeline contractors. Since the roadway project was the main driver of the schedule, funding, and construction, it was initially thought that a roadway contractor would be selected for the joint construction project. It was observed during the pre-bid meetings for both projects that pipeline contractors with qualifying experience were taking the lead as the general contractor and pavement contractors would be able subcontractors. This led to pipeline contractors eventually winning both construction projects with their relevant pipeline installation experience and the roadway portion of the projects were subcontracted to local paving and drainage contractors. These examples show the importance of determining qualifications for both pipeline and roadway contractors for a joint construction contract as discussed in the design phase of the project.

V. CONSTRUCTION

During construction, the project requires coordination on joint submittal or shop drawing review, quality assurance activities, delivery of materials, and inspection services for the joint project. Typically one agency takes the lead during construction phase. The lead agency for both segments of the Southwest Pipeline was the roadway project entity.

A. SUBMITTALS REVIEW

Prior to the start of either construction project, the contractor will begin to deliver submittals, test reports, and shop drawings based on the requirements of the contract documents. These may be different for each of the pipeline and roadway projects but the contractor will be required to submit them each to the entity involved in the projects. It is important to define the submittals review process in the contract documents and discuss this with the contractor prior to delivery of the initial submittals. There may also be different guidelines or submittal requirements for the separate projects for the same material type, test report, or safety plan for example.

The Southwest Pipeline project along Bonnie View Road, as previously discussed also has water and sewer line improvements along with the roadway and 96-inch pipeline. During the submittals review process, the contractor would submit a shop drawing that would encompass material for both projects. It is important to keep this in mind during the submittal review. Both the roadway and pipeline engineers should be responsible of reviewing the section of the submittals that pertain to their scope and a common response or separate response forms may be used to deliver comments back to the contractor. It was found to be more effective and less confusing to request that the contractor submit on both projects separately through one agency and avoid submitting one submittal that contains materials that need to be reviewed by both agencies.

B. QUALITY ASSURANCE

In any project, typically the contractor would perform quality control testing as required by the contract document and the lead agency would perform quality assurance tests to spot check the overall quality of the project. Proper quality control and assurance measures are necessary on any construction project, but it is especially important on a joint construction contract. Any quality issues that may arise on one project may impact the other. To avoid potential problems that could occur during construction, adequate quality control and assurance plans should be set in place for both the roadway and pipeline projects as discussed in the design phase. However, quality assurance needs to be well coordinated between both agencies. Both agencies may have different requirements and testing frequency for quality assurance activities performed by the agencies' own laboratory. This may require heavy coordination by two or three different testing labs to take samples for the same item.

The Bonnie View segment of the Southwest Pipeline project required significant quality control and quality assurance measures on both the roadway and pipeline projects especially on compaction that required the coordination among three different labs. Figure 3 below shows the backfilling operations on the pipeline. Both Dallas County and DWU also had their own testing labs to perform quality assurance for the entire project. This is an important consideration for a joint construction contract, since different entities may have their own guidelines that must be followed.

Figure 3. Backfilling Operations on the 96-inch Pipeline

C. MATERIAL DELIVERY

During construction of the roadway and pipeline projects, materials will constantly be delivered after the submittals are accepted by the engineer. Large diameter pipeline projects may require several miles of pipe joints to be delivered to site. In many cases, the pipe may be ready to be delivered but the site is not ready for delivery. This may be an issue that needs to be thought of during design and provisions for pipe storage should be considered. Storing the pipe material at the pipe factory may be an option until site conditions allow for delivery. The opposite scenario may occur depending on the phasing of construction and any milestones that have to be met. The pavement progress may be held by delays in the pipe production and delivery. It is critical to coordinate this early on during construction at the pre-construction meeting. This is especially important if there are two different pavement and pipeline contractors involved in the project. Figure 4 below shows the 96-inch steel pipeline joints being stored along Bonnie View Road prior to installation.

Figure 4. Southwest Pipeline Material Storage

D. INSPECTION SERVICES

During the construction phase of the roadway and pipeline projects, onsite inspection should be performed on a regular basis. Typically the public agency provides its own in house inspectors to inspect the job. In the case of a joint construction project, each agency would provide inspectors to supervise the construction of its portion of the project. Coordination among agencies' inspectors and project managers is required on a daily basis to coordinate quality assurance testing activities, regular construction meetings, reviewing and approving pay requests, and answering any RFIs from contractors. Inspectors were provided by the roadway project entity as well as the pipeline entity in both segments of the Southwest Pipeline Project.

VI. CONCLUSION

Joint construction contracts can be very challenging, but provide for several opportunities during the different phases of the project. The opportunities to save time, money, and resources to join two (or more) projects together are hard to ignore. So far, Southwest Pipeline has had three separate joint construction contracts over 32 miles of pipeline, with the possibilities for more in the future. This has saved time, resources, and overall construction cost for all agencies involved. However, the success of this project delivery method requires heavy coordination starting at the early planning phase of both projects to reap the benefits.

Challenges Associated with the Implementation of the Carlsbad Desalination Conveyance System

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Abstract

The San Diego County Water Authority (Water Authority), in conjunction with a private development team, is nearing completion with the construction of the Carlsbad Desalination Conveyance System (Conveyance System). A series of projects will deliver water from the new 54-mgd Carlsbad Desalination Plant to the Water Authority's Aqueduct System. The Conveyance System consists of four separate projects, three of which are being implemented through separate design-build (DB) contracts. The fourth is being implemented through a design-bid-build (DBB) contract. The fifth project, which is the desalination plant itself, is included herein for clarity, but is not considered as part of the conveyance facilities. The projects include the following:

- Carlsbad Desalination Plant – 50 mgd seawater desalination plant
- Carlsbad Desalination Product Water Conveyance Pipeline: (DB) Ten (10) miles of 54-inch steel pipeline with operating pressures up to 550 psi.
- Pipeline 3 Relining (DBB): Five (5) miles of existing 75-inch and 72-inch pipe are being relined with 69-inch and 67.75-inch steel liners.
- TOVWTP Improvement Project (DB): Approximately 1,500 feet of 54-inch steel pipeline, clearwell improvements, a flow control facility, and chemical injection facilities..
- San Marcos Vent Desal Modifications (DB): Includes connections between Pipelines 3 and 4 of the Water Authority's Second Aqueduct.

This paper addresses multiple challenges associated with the implementation of the Conveyance System. Specifically, it will address the challenge of coordinating design and construction of interrelated projects, handling simultaneous reviews of numerous pipeline headings, and establishing design criteria and resolving conflicting recommendations.

INTRODUCTION

The Water Authority was formed in 1943 to provide a supplemental supply of water to the San Diego region's civilian and military population and to meet expanded wartime activities. As shown in Figure 1, the Water Authority consists of 24 member agencies. Each agency purchases water from the Water Authority for retail distribution within their service areas.

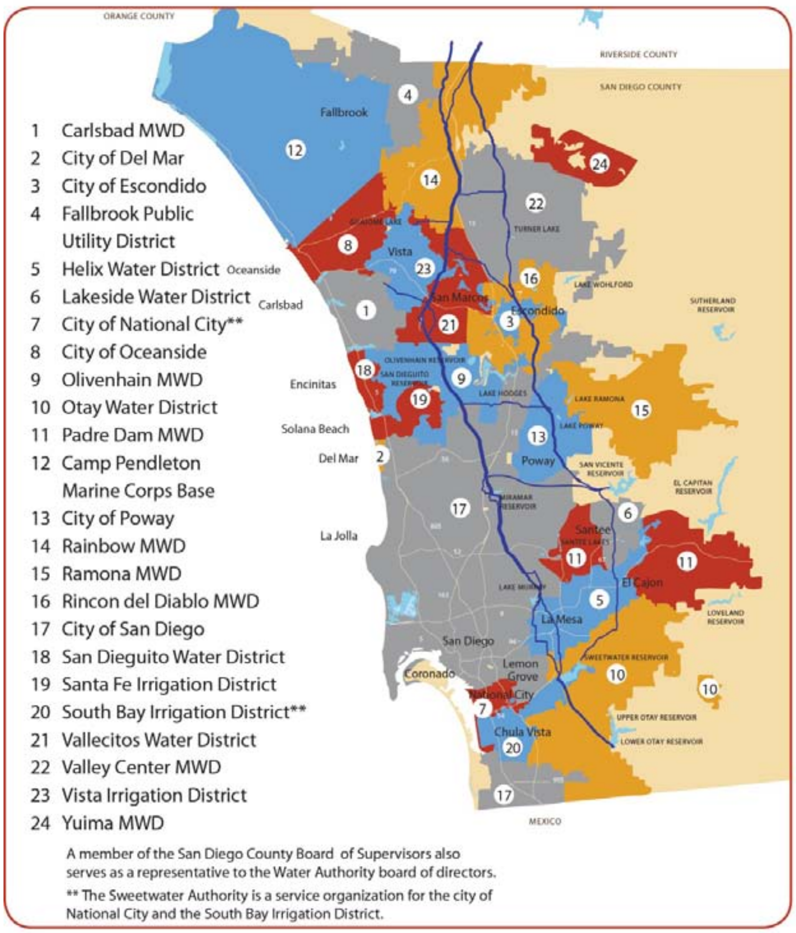

Figure 1. Water Authority service area and member agencies.

In 1947, San Diego began importing water from the Metropolitan Water District of Sothern California's (Metropolitan's) Colorado River Aqueduct (CRA) through a single pipeline to San Diego. To meet the demand of a growing population, the Water Authority constructed four additional pipelines between 1950 and the early 1980s to deliver water from Metropolitan's CRA and State Water Project Supplies. Metropolitan's two primary water sources are shown in Figure 2. (SDCWA 2011)

Figure 2. Metropolitan primary imported water sources.

Carlsbad Desalination Project. In 1991, 95 percent of the San Diego region's water supplies came from Metropolitan, making the region extremely vulnerable to water supply shortages. That year, an ongoing drought forced Metropolitan to cut deliveries to the San Diego region by 31 percent. As a result of that crisis, the Water Authority's Board of Directors approved a strategy to aggressively diversify the region's water supply portfolio by developing new local and imported water supplies.

Today, San Diego's imported water supplies consist of water purchases from Metropolitan, core water transfers from Imperial Irrigation District and canal lining projects that are transported through Metropolitan's conveyance facilities, and spot water transfers that are pursued on an as-needed basis to offset reductions in supplies from Metropolitan. This strategy has reduced its reliance on Metropolitan supplies to 45 percent. In addition, by 2020, local water supplies are projected to meet more than a third of the San Diego region's water demand.

In 2003, the Carlsbad Desalination Project, under development by Poseidon Resources, was incorporated into the Water Authority's Water Facility Master Plan. In 2010, the Water Authority entered into formal negotiations with Poseidon, and, on November 29, 2012, the Water Authority's Board of Directors voted to approve a Water Purchase Agreement (WPA) with Poseidon for the purchase of between 48,000 and 56,000 acre-feet (between 7 and 10 percent of the Water Authority's total annual deliveries) of desalinated water per year for 30 years. (SDCWA, October 2014)

With the plant expected to come on-line in the fall of 2015, the Water Authority, in conjunction with numerous consultants and contractors, embarked upon four additional projects to allow conveyance of water from the desalination plant into the Water Authority's Aqueduct System for delivery to member agencies. These projects make up the Carlsbad Desalination Conveyance System (Conveyance System) and include the following as shown on Figure 3:

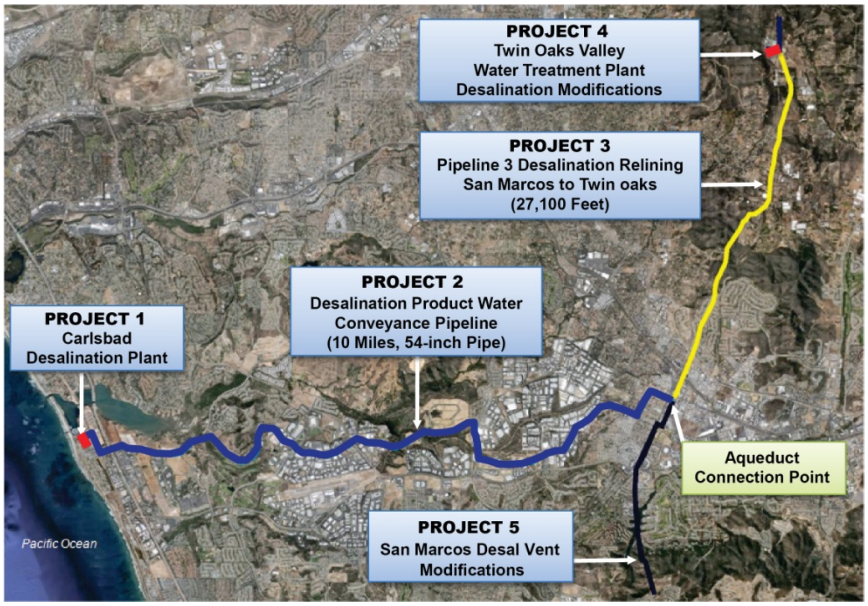

Figure 3. Conveyance system overview map.

- Project 1 (DB) - Carlsbad Desalination Plant (Desal Plant): 50 MGD Seawater Desalination Plant
- Project 2 (DB) - Carlsbad Desalination Product Water Conveyance Pipeline (Product Water Pipeline): Ten (10) miles of 54-inch steel pipeline with operating pressures up to 550 psi to deliver water from the Carlsbad Desalination Plant to the Water Authority's Pipeline 3 of the Second Aqueduct.
- Project 3 (DBB) - Pipeline 3 Relining: Five (5) miles of existing 75-inch and 72-inch pipe are being relined with 69-inch and 67.75-inch steel liners to repurpose the Water Authority's existing Pipeline 3 to convey desalinated water from the connection point with the Carlsbad Desalination Conveyance Pipeline to the TOVWTP.
- Project 4 (DB) - TOVWTP Improvement Project: Approximately 1,500 feet of 54-inch steel pipeline to convey water from Pipeline 3 to the TOVWTP clearwells. Additionally, clearwell improvements to thoroughly mix desalinated water with the water treatment plant flows, a treated water flow control facility improvement, and chemical injection facilities will be completed.
- Project 5 (DB) - San Marcos Vent Desal Modifications: Includes vent modifications and connections between Pipelines 3 and 4 of the Water Authority's Second Aqueduct to provide continuous delivery of treated water

to downstream Water Authority member agencies during relining of Pipeline 3 and after all pipeline interconnections are in place.

The Conveyance System operational schematic is shown on Figure 4.

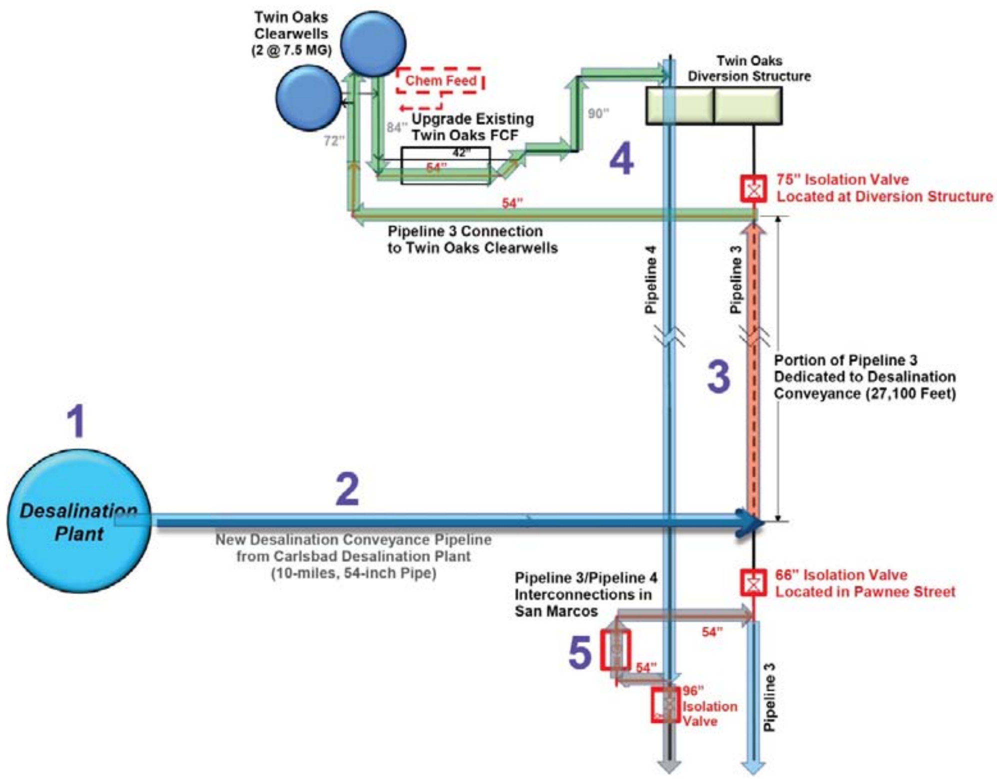

Figure 4. Conveyance system operational schematic.

Prior to execution of the WPA, to the Water Authority contracted with a consultant to perform fatal flaw review of the preliminary Conveyance Pipeline design documents, assist with the reviews during the development of pipeline design criteria, produce planning documents for improvements for the TOVWTP improvements, and prepare Service Contract Amendments with the TOVWTP operator to design-build and operate the improvement at the plant and ultimately serve as Owner's Representative for the design-build contracts related to the Product Water Pipeline and TOVWTP Improvements projects.

OWNER'S CHALLENGES COORDINATING AND EXECUTING MULTIPLE PROJECTS

During the design and construction of the Conveyance System, the Water Authority faced multiple challenges coordinating and executing the projects. Their primary goal was to ensure all project components were constructed, tested and operational by June 2015 in order to meet contractual obligations to take water from the Desalination Plant. In order to meet this goal, the Water Authority:

- Acted to ensure consistency between projects and facilitated resolution of conflicting consultant recommendations
- Coordinated contractor responsibilities for sequencing of construction, testing, disinfection and connections to ensure efficient implementation of project components

Design consistency between projects. Facilitating resolution of technical discrepancies and physical project interfaces between designs was one of the Water Authority's primary responsibilities. Meeting this challenge was key to ensuring the projects would operate harmoniously as a single system. A few of the inter-project design issues included project specific transient analyses, pipeline interface connection details, system supervisory control and data acquisition, system-wide cathodic protection, unique pumping and system operation considerations, and separate direct connections by two Water Authority member agencies. While individual design-build firms ultimately had responsibility for their respective project components, the Water Authority, through detailed reviews, comments, and coordination, ensured each component met overall project criteria.

In particular, the transient review affected every project component and was needed to define essential design criteria (e.g. ultimate steel pipe thickness, relief vent height at TOVWTP, and surge tank design at the Desal Plant). Each designer of record performed its own transient analysis. As a result, there were four separate analyses performed, each focused on the individual project components, and each with somewhat varying results. To ensure the Conveyance System was designed to meet all operating scenarios, the Water Authority retained their own transient expert to review each report and provide comments back to each analyst. Ultimately as a final independent verification, the Water Authority's expert ran an independent transient analysis confirming expected final system operation based on the individual project components/inputs.

Efficient implementation/sequencing of the project components. While the WPA identified completion milestones for the Poseidon and Water Authority-led projects, there were numerous milestones within each project tied to construction access/availability, hydrotest and disinfection activities, and most importantly coordination with ongoing operation of the Water Authority's Second Aqueduct and deliveries to its member agencies. Figure 5 illustrates a few examples of the construction and operational links that were coordinated between the projects.

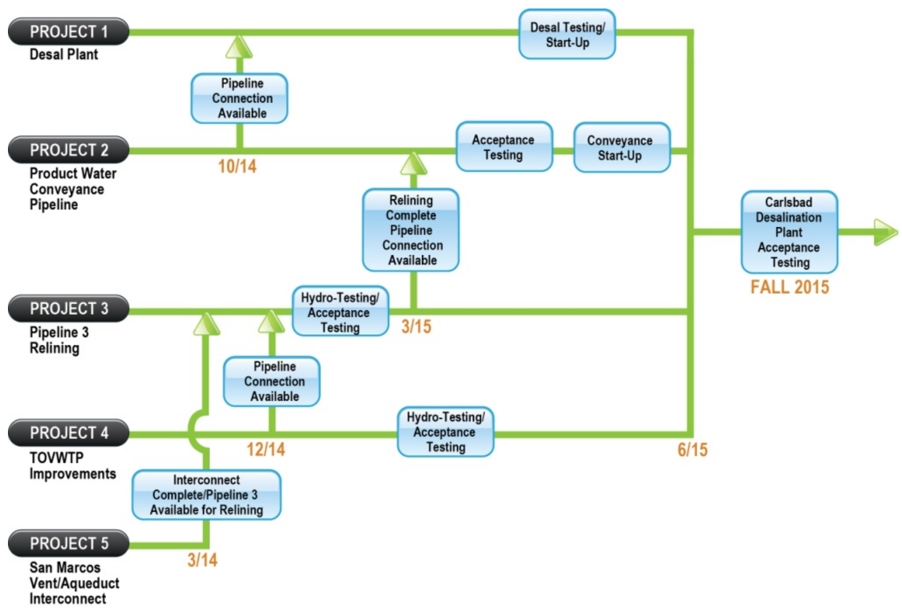

Figure 5. Conveyances construction and operational links.

As mentioned above, ensuring reliable aqueduct operations and member agency deliveries during construction and testing activities were essential for the Water Authority. Completion of the San Marcos Vents Desal Modifications project was the first step required to meet this challenge. The project included a downstream interconnection between the two treated water pipelines associated with the Conveyance System (Pipelines 3 and 4 of the Second Aqueduct) and a weir structure to boost pressure. Construction of the Pipeline 3 Relining project could not begin until the interconnection was functional allowing deliveries in both pipelines south of the relining work limits, which serve member agencies at different pressures.

From that point, the Water Authority coordinated several more system shutdowns to accommodate connections and testing associated with all the project segments. Meeting all schedule and work commitments during each shutdown was a key success factor for the entire project and required significant effort by the Water Authority and each contractor to coordinate.

PRODUCT WATER PIPELINE CHALLENGES

During the development and implementation of the Product Water Pipeline, a number of issues had to be addressed in order for the project to be successful for Poseidon and the Water Authority. Two major issues included:

- Leveraging resources, developing procedures, and obtaining stakeholder buy-in during the review of design packages.
- Developing consensus on methodology to analyze technical issues.

Design review process. The design –build of the Product Water Pipeline is being implemented by Poseidon. However, since the Water Authority will ultimately own, operate, and maintain the pipeline, it was responsible for ensuring the WPA contained an appropriate performance specification for the pipeline supplemented with Water Authority-developed design guidelines and standards for materials and construction. Once the WPA was executed, the Water Authority was required to modify its typical lead role to that of reviewer. The challenge for the Water Authority included:

- Limiting liability by not "directing" the design-builder on how to perform the design.
- Ensuring conformance to WPA documents through review of design packages and calculations.
- Coordinating reviews and design considerations with Operation and Maintenance staff.
- Retaining final "right of refusal" to accept the project upon its completion.

To initiate construction as soon as possible, the design-builder initially divided the project into six design packages, which ultimately expanded to ten packages as shown in Figure 6, and multiple pipe calculation submittals. Each design package was submitted as draft and final. The Water Authority's challenge included reviewing each submittal to ensure conformance with the WPA (e.g.: Water Authority design criteria and standards, industry and project-specific standards, and to ensure operability and maintainability for the Water Authority). Over the course of the project, the Water Authority and its consultant reviewed multiple packages simultaneously and submitted comments to the Water Authority within two weeks of receipt of each design package so that the Water Authority could meet the turnaround requirements set forth in the WPA.

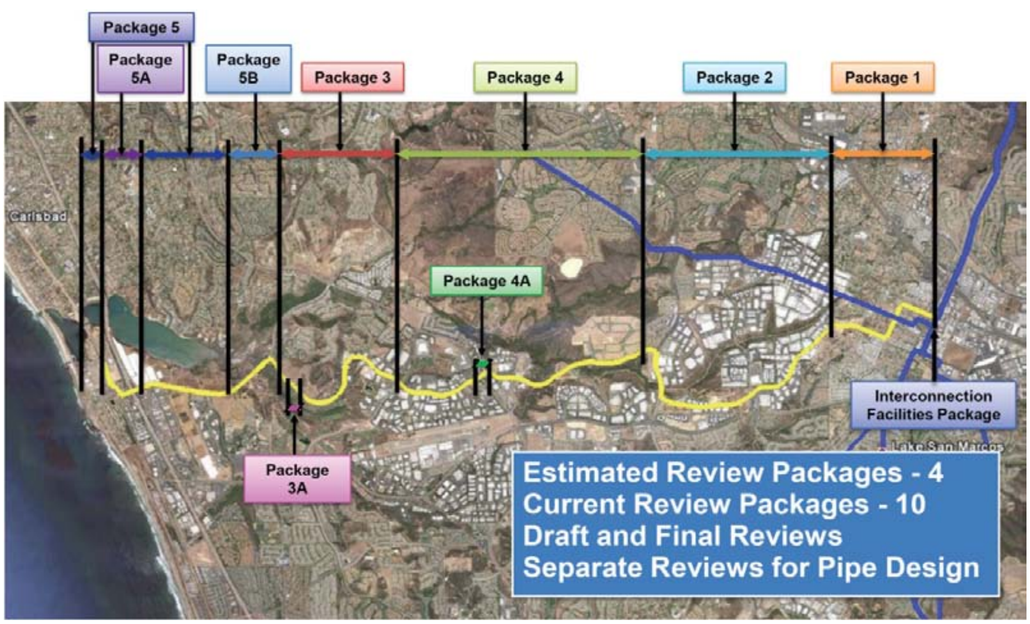

Figure 6. Conveyance pipeline submittal packages.

To accomplish theses tasks, a dedicated consultant review team was assembled, consisting of personnel specializing in Water Authority standards, pipeline design, operation and maintenance (O&M), and electrical and structural disciplines. Once the individual reviews were completed, to the team met to verify applicability to the project scope, ensure of the comment, and screen out duplicate and/or conflicting comments.

Once reviews were complete, tabled comments integrated with Water Authority staff comments. Once the comments were combined, the comments were formatted in two separate tables. The first table included comments that required action in accordance with the WPA; the second table included Water Authority preferences not clearly covered in the WPA and "clean up items" for consideration by the design-build team. Dedicated consultant staff was provided on-site at the Water Authority's office to assist with assembling the comments.

Each comment was then addressed by the design-builder in the comment table and incorporated into the design package. During subsequent reviews, responses were verified to ensure the comment was adequately addressed or if further action was required.

This process was similarly and simultaneously utilized on the design reviews for the TOVWTP Improvement Project.

Developing consensus on technical issues. During the fatal flaw review period, the Water Authority's team reviewed the design-builder's pipe design criteria. Due to the high pressures (approximately double the maximum pressure in the Water Authority's aqueduct system), large diameter, and location of the pipeline in heavily traveled roads, business parks, and residential areas, it was recommended that the pipeline design be conducted with a higher level of analysis than would be afforded a typical water transmission main. Of particular concern were design issues related to the long-term reliability and serviceability of the pipeline.

To develop a process to address this concern, facilitate exchange of information, resolve conflicting expert recommendations, and ensure the appropriate criteria was included in the WPA, the Water Authority:

- Held workshop meetings between experts to discuss review comments, explore leading questions and brainstorm design methodology.
- Provided review comments to the design-builder in the form of leading questions being cautious not to direct the design-builder.

This process was utilized to resolve two technical challenges critical to the long-term reliability and serviceability of the Product Water Pipeline:

- Allowable design hoop stresses
- Joint type and analysis

Allowable Steel Hoop Stresses (Tetra Tech, August 2012)

Prior to development of the WPA, the design-builder's design included shop-applied mortar-lined pipe with an allowable design hoop stress at working pressure (plus a 50 psi allowance for surge) of 50 percent of steel yield (20 or 21 ksi based on Grade 40- or 42-ksi steel). Water Authority standards limit the allowable stress to 18 ksi for mortar-lined pipe.

The Water Authority provided comments to the design-builder on their proposed allowable stress. The comment required evaluation of the long–term performance of the mortar lining in order for the Water Authority to relax its standards. In a workshop, the Water Authority and the design-builder discussed the requirement and agreed that a stress-strain analysis of the steel and mortar lining would be appropriate to satisfy the requirement.

Using the stress-strain relationship for mortar included in AWWA C304 and shown on Figure 7, the design-builder performed the analysis. The results showed a maximum strain of 825 micro-strain, or approximately 70 percent of the value at which visible cracking was estimated to occur, under surge conditions.

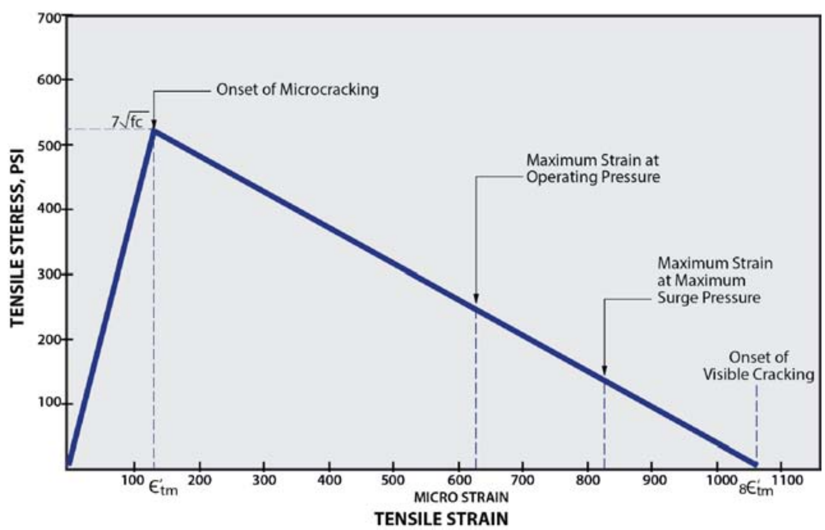

Figure 7. AWWA C304 stress-strain diagram for mortar (AWWA 2007).

After review of the initial evaluation, several additional effects likely to cause the strain to be higher were identified and reviewed with the design-builder.

- Shrinkage strains due to drying of the mortar and along spiral welds
- Tensile softening of the cement-mortar lining after shrinkage and application of the first set of internal pressure loads
- Internal pressure load cycling (inhibiting autogenous healing of the lining)
- Performance of field-placed cement mortar at pipe joints

After a meeting between the Water Authority and the design-builder to review these issues a decision was made to adhere to the Water Authority Standards, of limiting the allowable steel stresses to 18 ksi and 21 ksi for working and surge pressures, respectively.

Joint Stress Analysis (Tetra Tech, November 20, 2012)

The initial joint stress analysis proposed by the design-builder involved the use of ASME joint efficiencies for single and double lap-welded joints. Butt-welded joints were not being considered for use on the project. The Water Authority's consultant initial recommendation included the use of butt-welded joints above certain thresholds; however, the design-builder preferred lap joints to maximize flexibility for fit-up in the field.

To resolve these differences of expert opinions, the Water Authority held a series of workshops to discuss informally and formulate a plan of action. Theses workshops resulted in agreement on a methodology for stress analysis to evaluate the performance of double lap welded joints for the project. The details of the analysis were developed by the design-builder and reviewed by the Water Authority's consultant.

Of particular interest was the evaluation of the bending stresses in the lap-welded joints. This procedure, depicted in Figure 8, included evaluation of the bending moment and shear forces across the lap joint, as well as bending and shear stresses at the weldment.

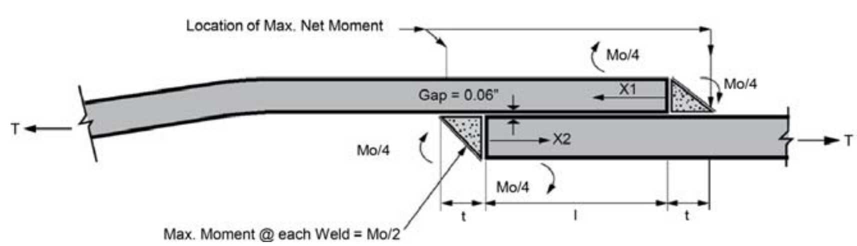

Figure 8. Joint stress analysis.

The analysis, based on the Roark's Formulas for Stress and Strain, superimposed moments at the bell and spigot ends of the joint. The maximum net moment at the spigot and bell were calculated using the appropriate equations from the Roark's Formulas.

The results of the analysis showed that double lap-welded joints were acceptable for the project in all locations. In addition, by performing this detailed analysis, it allowed the Water Authority to make informed decisions to allow stresses at the joints to exceed allowable limits under certain operating scenarios, particularly seismic. By doing this, the Water Authority was able to avoid requiring a thickened cylinder over the entire pipe can, to keep joint stresses below established thresholds.

SUMMARY

The Water Authority faced multiple challenges and issues in coordinating and executing the projects to ensure their completion by June 2015. In order to address these challenges and meet contractual obligations to take water from the Desalination Plant the Water Authority acted to ensure consistency between projects, facilitated resolution of conflicting recommendations, and coordinated contractor responsibilities for construction and start-up sequencing.

During the development and implementation of the Product Water Pipeline, a number of challenges had to be met in order for the project to be successful for the design-builder and the Water Authority. Major challenges included leveraging resources, developing procedures, obtaining stakeholder buy-in, and developing consensus on methodology to analyze technical issues

Design review challenges were met by assembling a dedicated review team consisting of personnel specializing in Water Authority standards, pipeline design, operation and maintenance (O&M), and electrical and structural disciplines and by following a detailed process by which comments could be communicated, tracked and verified.

Technical challenges and differing expert opinions were resolved through workshop meetings between experts to discuss review comments, brainstorm design methodology and providing review comments to the design-builder in the form of leading questions being cautious not to direct the design-builder.

In conclusion, success of the project relied on to work together to develop mutually acceptable project criteria to enable the design and construction of the Carlsbad Conveyance System within the requirement set forth in the WPA.

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New Day, New Conflict (The Challenges of Water/Wastewater Design for a Multi-Billion Dollar Highway Design-Build Project)

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Abstract

Large urban highway widening projects will have many water and wastewater pipeline conflicts. Those conflicts are caused by various aspects of the highway project including the following: road pavement widening through lane addition, new/realigned storm sewer, new bridges, new retaining walls and proposed grading changes. This paper presents the challenges and advantages of water and wastewater pipeline relocation design as part of the design-build large urban highway project with Public-Private Partnership (3P) funding and project delivery. This paper also addresses possible ways to improve the process for future work. The utility relocation portion of the project consisted of eliminating all possible water and wastewater conflicts with the aforementioned design components of the roadway. The project scope consisted of three large highway corridors in Tarrant County, Texas. Tarrant County is the third most populous county in Texas according to the 2010 U.S. Census and the county seat is Fort Worth, TX. Approximately 150 water and wastewater conflicts were analyzed throughout the project. This equates to 106 miles (170 kilometers) of water, wastewater and franchise utilities impacted by the project. The total construction cost for these three urban highway reconstruction projects was $4.1 billion.

INTRODUCTION

This paper presents the challenges and advantages of water and wastewater pipeline relocation design as part of the design-build large urban highway project with Public-Private Partnership (3P) funding and project delivery. This paper also addresses possible ways to improve the process for future work. The utility relocation portion of the project consisted of eliminating all possible water and wastewater conflicts with the aforementioned design components of the roadway. The project scope consisted of three large highway corridors in Tarrant County, Texas. Tarrant County is the third most populous county in Texas according to the 2010 U.S. Census and the county seat is Fort Worth, TX. Approximately 150 water and wastewater conflicts were analyzed throughout the project. This equates to 106 miles (170 kilometers) of water, wastewater and franchise utilities impacted by the project. The total construction cost for these three urban highway reconstruction projects was $4.1 billion. Segment 1 of the North Tarrant Express (NTE) project is the portion of IH-820 from I-35W to the IH-820 northeast loop interchange (Figure 1). Segment 2 is the portion of SH 183 from the IH-820 northeast loop interchange to the SH 121/SH 183 split (see Figure 1). Segment 3A of the North Tarrant Express (NTE) project is the portion of IH-35W between downtown Fort Worth and north loop of IH-820. This specific segment is the most congested highway corridor in Tarrant County and ranks 8th overall in Texas according to the 2014 Texas Department of Transportation (TxDOT) 100 most congested highways (see Figure 1). The map below provides an overview of the project limits within Tarrant County:

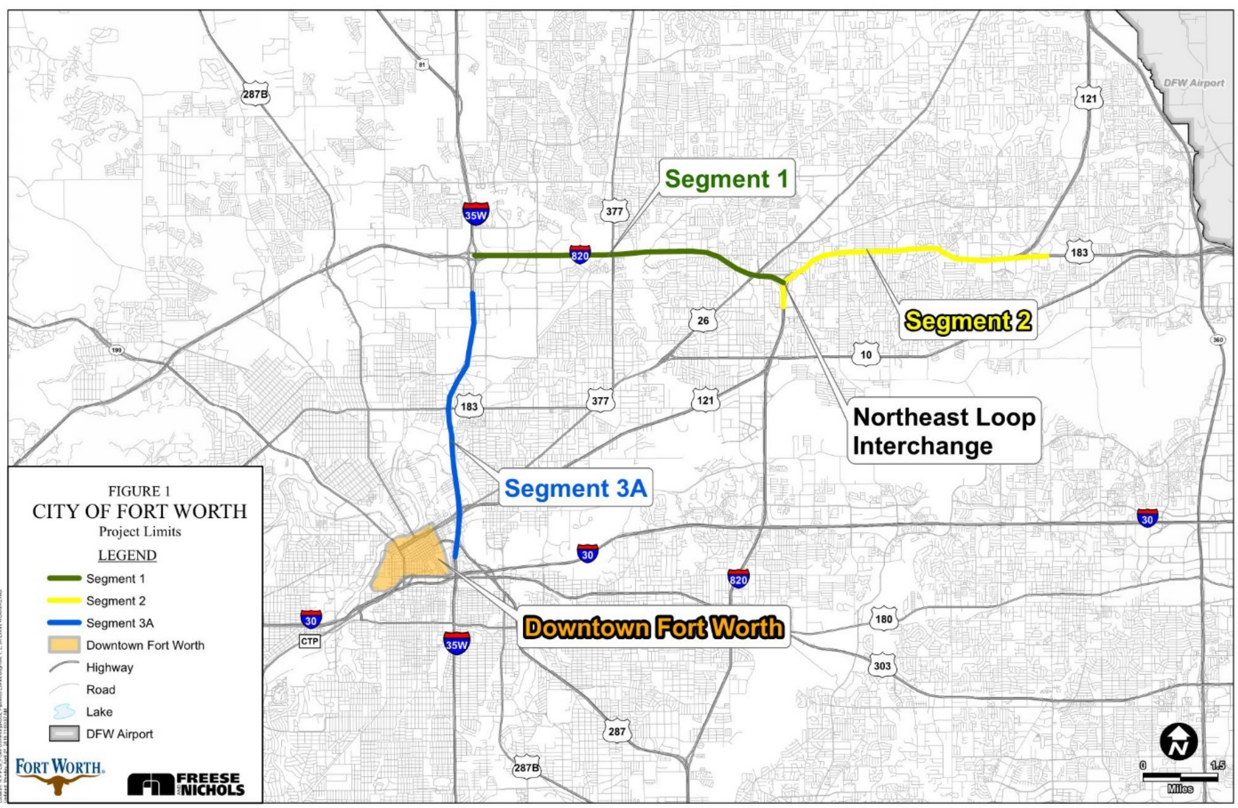

Figure 1 (Segment 1, 2 & 3A of the North Tarrant Express)

The main aspect of the water and wastewater pipeline relocations that made them unique was the design-build project delivery method. The design-build project delivery brought with it a fast paced schedule, constantly evolving priorities/timeline, changes in highway design and multiple design review entities. As the need for massive highway reconstruction projects in highly developed corridors continues to rise, the design-build project delivery methodology for these projects will be employed more over the years to meet these challenges.

The challenges with a design-build type project are the constantly changing conflict priorities and schedules. This is due largely to the fact that the water/wastewater design is done concurrently with the design of the rest of the project; specifically, roadway, storm drain, retaining wall and bridge design. Often times the design or construction of one of the other elements becomes an unforeseen condition resulting in conflict reprioritization and possibly redesign.

This paper presents the basics of the design-build project delivery methodology, the challenges and advantages of design-build methodology and the specific lessons learned from this project.

PROJECT DELIVERY

This section discusses the general application and nuances of the project delivery and team for this design-build project. The main topics that had the most impact on the project delivery were stakeholder influence, the implementation of the design-build project methodology and the design and review process.

Stakeholder Influence. Due to the enormity of this project there are numerous stakeholders with varying stakes in the project. This varies from the private developer who is the financial backing and implementation for the project as a whole to the local business owner who is losing a significant amount of their highway frontage property and forced to relocate their parking lot or possibly their entire business. Obviously, the greater the stake in the project the more influence a given stakeholder can wield during the project life cycle. The main stakeholders for these types of projects are the developer (NTE Mobility Partners), State infrastructure governing agency (TxDOT), franchise utilities (gas, electric, cable and fiber), and municipal (City of Fort Worth and the City of Hurst) utilities (water, sewer and storm drain). On top of these major stakeholders, you also have public impatience with traffic delays putting additional pressure on the schedule. The water and sewer design engineer was in charge of the design of those utilities from initial conflict analysis to final design. The overall utility relocation manager was responsible for the overall utility coordination of the project between both municipal and franchise utilities from initial conflict detection through construction.

Along with the varying influences, each stakeholder has their own interests. On 3P projects it can be cumbersome because you often have multiple organizations with competing interests. For instance, there are three legs to the project “stool” (budget, schedule, and quality). The utility owner is primarily concerned about quality and minimum customer disruptions. The developer is primarily concerned with schedule and budget of the overall program. These competing interests can make the design process difficult as there are numerous solutions designing a resolution to a specific utility conflict. With multiple entities reviewing the design, you receive several valid and constructive comments where neither side is technically incorrect. What matters is which leg of the stool you are focused on.

Often times these interests will clash. One particular example was on an old large sanitary sewer line in a section of the highway that was adding about 14 feet (4.3 meters) of fill on top of the existing conditions. The utility owner insisted that this needed to be replaced due to the additional fill; however, the developer and utility coordinator did not think it would make a difference due to the geotechnical profile of the area. The utility owner saw this as a chance to replace aging infrastructure at no cost to them due to the agreement with the developer, while the developer saw this as somewhere they could eliminate time and money from the overall project. The design engineer, as the consultants, was asked by the utility coordinator to evaluate the proposed conditions. The design engineer found that because the original pipe was installed in concrete casing pipe and was originally installed by other than open cut methods through a shale layer. That the pipe would not fail due to the additional loading as a result of the highway project.

Design Build Project Implementation. This project was not your typical design-build project due in large part to the size and scope of the infrastructure. The project as a whole is a design-build project but the individual pieces resembled more of a fast paced design-bid-build (DBB) project that also had several other DBB projects going on in the same area in parallel. The project was divided up into multiple packages so that the construction of the utility relocations could commence in a staggered fashion allowing for a condensed schedule and the ability to adjust sequencing to mirror the highway progression. That being said, incorporating this as one major design-build project allowed for the communication and coordination between disciplines before design finalization. Imagine coordinating with 7 different design disciplines over 13 miles (21 kilometers) of highway in a large urban area without the flexibility and coordination provided by the design-build project methodology.

Design & Review Process. The design and review process for this project was well developed, if not possibly over developed. The design work for infrastructure, storm drain, franchise utilities and municipal utilities was all sub-contracted to different consulting firms as a way to condense schedule by allowing the design processes to take place simultaneously. This, as you can imagine, creates a serious coordination challenge as the iterations change slightly, creating or possibly eliminating utility conflicts. For instance, a bridge pier shifts 5 feet (1.5 meters) and is now in direct conflict with a 12-inch water line that was not previously designated for relocation. Or the water line has shifted to make room for other utilities within the ROW, changing its interaction with the storm drain and causing a conflict. Like most of the other conflicts that arose these scenarios were reviewed through conflict design resolution. This was accomplished through cost benefit analysis and value engineering principles by comparing the alternatives available and the effects on cost and schedule. These scenarios, as well as many others, played themselves out numerous times throughout the projects.

The review process had several design review entities reviewing each set of plans submitted. Due to the amount of review that was needed to be accomplished by the Municipality, the developer, as part of their agreement, paid for a third party reviewer to help the municipality with their utilities plan review. This was done as a way to help ensure that the process did not get bottle necked during the design and review process.

As part of the pipeline review process, a comment log was used to help track comments made as well as providing the consultant a method of responding to minor comments without having to spend the time in a meeting. The comment log, while cumbersome at times, proved as a useful tool to track design changes and the decisions behind those changes. In addition to the design reviews, there were meetings held every two weeks between TxDOT, the developer, and the City to coordinate the design process, legal paper work, and construction issues. The design were reviewed at the following stages; 30%, 60%, and 90%. As comment logs were received, the design engineer developed an internal design checklist specific to this project based on the municipality's preferences shown in the comment log as well as the requirements of TxDOT. This was a living document that updated as packages were reviewed and constructed.

As mentioned earlier with all of these designs taking place in parallel the need for coordination was key. For this particular project a large portion of the information sharing came via an FTP site where the utility coordinator would receive updated plans from the various design disciplines and post them to the site for incorporation in the next iteration of design. The utility coordinator also kept a living map of existing and proposed utilities contained within the project limits. Then, if issues were noticed in the drawings or on the map due to a design change to a given discipline, the disciplines would coordinate to resolve the conflict using value engineering principles to help guide their decisions.

CHALLENGES

As with all projects there are challenges. These large fast pace design-build urban highway projects bring a new set of challenges that are unique. These challenges include working around old infrastructure, changes in other design components, changing priorities, balancing stakeholder interests, fast-paced schedule and data management.

Insufficient Record Data. These urban highways were built in the 1960's. Most of the utilities were also built around this same time frame. Some of the utilities are older, including a few that are close to 100 years old. With this old infrastructure it can be difficult to find good record data leaving the Subsurface Utility Engineering (SUE) group with a large number of unknowns. These unknowns led to errors in the initial SUE. The SUE was performed at quality level ranging from D to A depending on field conditions, ability to tone the utility line and information available. Multiple SUE firms performed the work and there were times when conflicting data was found. There were many times when additional survey and SUE investigations were required to better locate the existing utilities. This was especially apparent when SUE became necessary outside the highway project limits. A lot of times the connection point or water shut-off valve was outside of the SUE project limits and these points needed to be verified to determine which water costumers would be effected by the construction of the proposed line. Also, there were instances where the proposed utility needed to be placed just outside of the highway project limits and there was no survey or SUE data done.

Parallel Design. Changes in other design components of the project was another significant challenge. These design components include storm drain systems, retaining walls, sound walls, and other utilities. With the fast pace of this project, some of these components are designed at the same time as the utilities. So when a change is made to one of the design components, it can have a have a ripple effect on the design of other components. This causes an almost constant state of flux for the horizontal and vertical conflicts for the proposed utilities. Interim stages of construction also need to be considered during design. For instance, an additional 20 feet (6.1 meters) of soil is placed over a proposed pipeline alignment that is not part of existing or ultimate grading conditions. Also, infrastructure that were once proposed utilities on plans may have been constructed sooner than anticipated and are existing rather than proposed when installation begins. This has caused constructability issues that were not considered during the design process. An example of this was when the design of a water line was finished and approved by the stake holders, a new proposed sound wall conflict became known with the water line. The location of this sound wall was not known when the water line was being designed. Both the proposed water line and sound wall had to be adjusted to correct the conflict. The water line was shifted to miss the drilled pier foundation of the sound wall and the sound wall changed the separation distance between bridge piers to help the water line design.

Figure 2 Building along Proposed Alignment

Fast Paced Schedule. With this fast paced schedule, not only do the design components change but the priorities also change. With all projects there are unanticipated schedule delays that can affect the overall project schedule. When this happens the whole project schedule may need to be adjusted to keep things moving and priorities change. This may include work that was originally planned to be constructed on the east side of the highway and cannot be done due to lack of environmental clearance or ROW clearance. ROW clearance was handled by the developer of the project. This occurred on Segment 3A of the project and resulted in changing the designs focus to the west side of the highway so that the project may move forward. For the utility design team there are times where these changes in priority come so fast it is a challenge to be able to focus on the item with the highest priority. Pressure is also placed when considering the scope of the utility relocations portion of the project relative to the overall project, a $228M component of a $4.1 billion project. The engineer also gains a grasp of the fast paced schedule on site visits. For instance, a walk along the proposed alignment may bring about obstacles that make it impossible to walk the whole alignment. In the picture below an educational building is scheduled for demolition. The magnitude of the schedule of the project is readily apparent with the knowledge that this building will be demolished within a month (see Figure 2).Even with the fast pace of the project, the engineer is still held to the same design standards as any other project and it is the engineer's responsibility to ensure that public safety is always the priority.

Balancing Stakeholder Interests. One of the biggest challenges was balancing stakeholder interests. To help combat this challenge, the stakeholders were able to review the design and make comments on the design. These comments were logged and required a written response to every comment. In some cases a comment resolution meeting was required to review or discuss these comments. With multiple reviews, the requirements for plan preparation evolved. As part of this evolution, the plan sheets became very detailed and busy, especially after several design iterations of packages. While most of these comments were constructive, many were just preferences on how the design plans were to look. The design checklist evolved as these preferences had changed throughout the project.

Data Management. With multiple comment logs, changes in highway design, and updated SUE information, it became a significant challenge to keep up with all of the data. One benefit of having multiple reviews is that other design teams can also check that the most up to date information is used. Another issue that arose several times is that the survey would show something different than another designers which should not be the case since they were both pulling from the same FTP site. This would cause an issue because both design teams would be designing around the vertical and horizontal conflicts shown in their designs.

ADVANTAGES OF DESIGN-BUILD METHODOLOGY

There are several key advantages that the design-build methodology brings to the table for the implementation of large urban highway widening projects.

Fast Paced Schedule. One of the main benefits of the design-build project delivery method is the expedited schedule. It allows for multiple design entities and various stakeholders to collaborate with each other better than a typical DBB project. One of the main distinctions for this can be seen with the roadway design. In a DBB project, the roadway design is designed to a 100% level. Once the project is completely designed, the utility relocation portion of the design begins based on the final drawings prepared from the roadway designer. In a design-build project, the roadway design is typically designed to a 90% level when the utility relocation design portion of the project commences. Not only does this decrease the overall schedule of the project but it allows for conflict analysis with the proposed roadway system before that portion of the project advertises. The schedule benefits of design-build can also be seen when you juxtapose two recent projects in Tarrant County: The Chisolm Trail Parkway and Segment 3A of the NTE. The Chisolm Trail Parkway is a 27.6 mile (44.4 kilometer) toll road that is mainly rural with a couple of urban segments. The project delivery method chosen for this project was DBB. The initial route was selected back in 1985 and the roadway was first opened for traffic in 2014. That amounts to a total of 29 years from initial concept to substantial completion of the project. Segment 3A of the North Tarrant Express is 6.5 miles (10.5 kilometers) of completely urban highway through the heart of Fort Worth. Segment 3A of the North Tarrant Express was conditionally awarded to North Tarrant Infrastructure in 2009 and has used the design-build project delivery methodology. Substantial completion is currently estimated for 2018. The schedule contrast is clear. The Chisolm Trail Parkway took 29 years from initial concept to substantial completion and Segment 3A of the North Tarrant Express is on schedule to meet the 9 year goal of substantial completion from initial concept. There are many other factors that play into the complexity of this comparison; however, the schedule benefits can be seen to some extent.

From a strictly design perspective, the fast-paced schedule can help immensely. Key design decisions are fresh on everyone's mind and project communication moves at a fevered pitch. Standard processes such as design checklists remain applicable throughout the design schedule and leave little room for code and regulation change that may affect current design methodologies. If a project schedule is delayed or put on hold it could open the partially designed segments of the project to substantial rework as a result of code or regulation changes. For this project, that was not a concern.

Figure 3 Proposed Bore Across Highway

Communication and Coordination. There are many different stakeholders in large urban highway planning, design and construction. Although the coordination efforts between parties can be daunting at times for a design-build project, a well-structured communication plan can allow for purposeful collaboration between all parties. Comment logs are a great way to allow for input from multiple stakeholders during the design process. It documents decisions made in the past and helps streamline all design comments for a specific pipeline into one concise decision log. If a certain stakeholder changes their mind about a particular design, the documentation has been made and a foundation has been laid for purposeful discussion without "hear say" debate.

Conflict Detection. Another benefit of the design-build project delivery method is the ability to detect conflicts prior to design finalization. As mentioned previously, the utility relocation portion of the design begins when the roadway design has reached a 90% level. This allows utility and roadway stakeholders and designers the ability to collaborate on conflicts and resolve them before construction. For example, a storm drain grade adjustment can save a lot of headache for a sewer crossing that is very limited in vertical alignment potential. A solution can be reached before permanent infrastructure is installed and before more costly remediation is required. This is where the multiple entities of design review prove their worth as each different entity is looking for their specific interests and overall catching most if not all of the major possible conflicts between updated/revised designs.

Opportunities for Betterment. For large urban highway projects like this, the utility owner may have the ability to improve their infrastructure through betterment. With large equipment on site, multiple contractors on site and mobilized, and lots of dirt moving, the utility owner may have the opportunity to upgrade lines during highway construction even if they may not be in direct conflict. There may be savings opportunities for the utility improvement project if the utility contractors are already mobilized around the construction area. Portions of the highway may be out of service during the construction that may remove or discount the traffic control and pavement rehabilitation bid item requirements from a specific utility project.

LESSONS LEARNED

As with every venture in life there are always things you learned. Some of these lessons are learned through experiencing a successful aspect of a project and some are learned through struggles. Either way they provide an opportunity to learn from your experiences and grow both as an individual and a profession. This project was no exception. There were several successful strategies or tools employed by various members of the project team and on the flip side there were strategies and tools implemented that were less than successful. Below we discuss both and how we can improve on them moving forward.

Design Checklist. While the internal design checklist was developed from experience with the municipality and TxDOT, It would have been beneficial to get buy-in from all the reviewing entities. This would have helped to get consensus or at least reduce conflicting design comments that were received on various packages and helped reduce turnaround time because the decision on how to move forward would have already been established. The comment log was a very successful tool in tracking and compiling all of the comments from the multiple design review entities. It was an excellent way to receive review input as well as respond to a review input while eliminating the need for dozens of review meetings throughout the project.

Data Management. One of the main difficulties in every project is communication. That only becomes more difficult the more entities you have designing and working in the same construction space. A large part of that coordination is availability of up to date design info. The data management for this project was accomplished by posting updated designs to an FTP site. This was a somewhat effective way to distribute up to date design information but the issue that arose was when you had partial submittals of plans. A smoother more streamlined process to improve the data management of a project like this would be to use a cloud type server. There are products out there that are built specifically for work sharing, ensuring project continuity and providing dynamic feedback through a cloud type file storage program. This program would allow individual designers to check out and check in files providing almost real time updates to the base files that the other designers are working from. This would help eliminate the need to comb through multiple pages or design only to find out that the applicable design is not affected or worse that you were reviewing an out of date submittal. It would also be an efficient way to ensure the most up to date files are in everyone’s hands which can be difficult with a project that has such a fast paced schedule.

Subsurface Utility Engineering. Another issue that arose was the coordination/availability of the SUE firm. One way to help with the accessibility issue of the SUE firm would be to have multiple firms focus on specific parts of the project (i.e. subsurface, topo, etc.), or even specific utilities to track down and research. This way could be best implemented along with the base file in the cloud storage scenario discussed earlier to help with the data management aspect of the project. Using one living document would also help to ensure that everyone is working off the same data. Another way to help with the survey and SUE would be to add an SUE coordinator to facilitate the requests from the designers for additional SUE or survey information.

CONCLUSION

As the need for future urban highway expansion continues across the nation, 3P design-build will continue to be a valuable project delivery method for use in coordinating pipeline relocations and many other aspects of the project. This paper presented specific challenges and advantages of the DB delivery method for three different large urban highway widening projects. This specific challenges discussed were; availability of record data, parallel multi-discipline design, fast paced schedule, balancing stakeholder interests and data management. The advantages presented in this paper were; fast paced schedule, communication and coordination, conflict detection and betterment opportunities. The paper also ventured to show areas of improvements in the lesson learned section. The areas focused on in the lessons learned section were; development of a design checklist, efficient data management and accessibility/reliability of subsurface utility engineering. Armed with the basics of the design-build process and awareness of the challenges, the engineer can leverage the advantages of design-build and lessons learned from this paper to deliver quality pipeline relocation projects in the future.

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Ductile Iron or Welded Steel? A Comparative Analysis between Pipe Materials for the Replacement of a Large Diameter Transmission Main

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Abstract

When a pipe reaches 60-inches in diameter, there are few materials of choice for designing a water transmission main. When the use of prestressed concrete cylinder pipe (PCCP) or bar-wrapped concrete pipe (BWCP) is not allowed, the choices become even more limited. Ductile iron is arguably the pipe material most recognized and utilized by municipalities for their potable water infrastructure (on the East Coast), especially for smaller diameter lines; however, at 60-inches in diameter, PCCP and spiral-welded steel pipe ("welded steel") are more prevalent. When adding in the requirement for a full cathodic protection system, an analysis is required for selection of an appropriate material for the project. So, under what circumstances does the scale tip in favor of ductile iron at these larger diameters? How about for welded steel? This presentation focuses on reviewing the characteristics, advantages and disadvantages to using both pipe materials on large diameter transmission main applications through the lens of a ductile iron/welded steel materials analysis that was completed for a 60-inch diameter transmission main for the Washington Suburban Sanitary Commission (WSSC).

OVERVIEW OF THE WASHINGTON SUBURBAN SANITARY COMMISSION

The Washington Suburban Sanitary Commission (WSSC) is one of the largest water and sewer utilities in the country, serving close to two million customers over a 1,000 mile area within the Maryland counties of Montgomery and Prince George's. WSSC owns and maintains several large water and wastewater treatment facilities and has over 5,500 miles of potable water lines and over 5,400 miles of sanitary sewer infrastructure. Of the Commission's 5,500 miles of potable water lines, approximately 350 miles are comprised of large diameter prestressed concrete cylinder pipe (PCCP) transmission mains, some of which have encountered significant failures in recent years and have reached the end of their useful life. Within its overall Capital Improvements Program (CIP), WSSC has a Large Diameter Water Pipe Replacement Program in place that is aimed at systematically repairing and/or replacing these mains.

SOUTH ADELPHI 60-INCH DIAMETER TRANSMISSION MAIN REPLACEMENT

In support of WSSC's Replacement Program, O'Brien and Gere was contracted to provide design services, including a detailed pipe material analysis, for the replacement of approximately one mile of WSSC's existing South Adelphi 54-inch PCCP main with a new 60-inch pipeline (Figure 1). The existing main is a critical piece of WSSC's infrastructure, serving as one of two suction lines that feed the Commission's Central Avenue Pumping Station, which in turn provides potable water to much of Prince George's County. The section of main scheduled to be replaced had experienced multiple breaks in recent years, resulting in several carbon fiber-reinforced polymer repairs, which limited the redundancy to the Pumping Station.

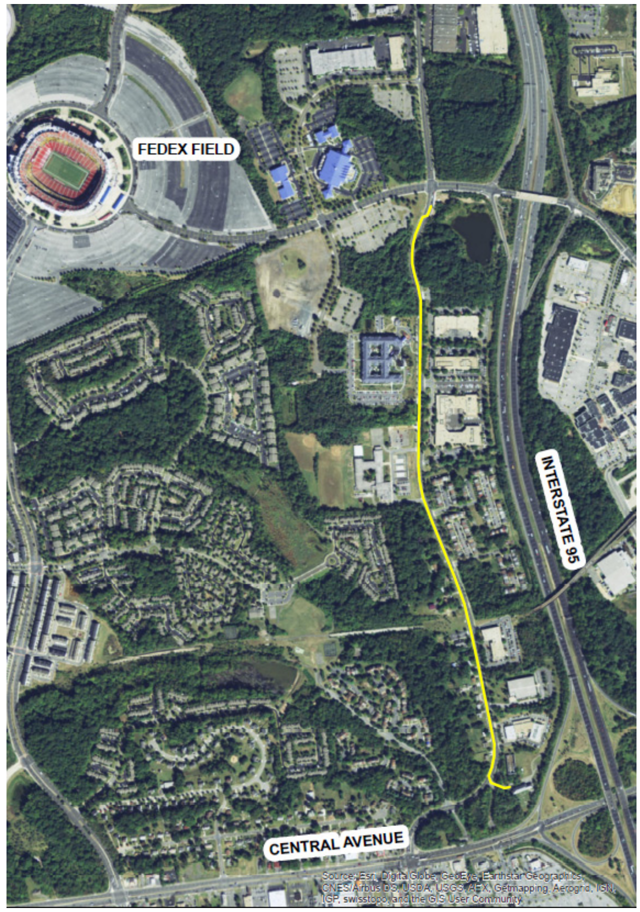

Figure 1. Proposed alignment for 60-inch replacement main.

INITIAL DESIGN

Before initiating the pipeline materials analysis, a review of available as-built documentation and an initial site walk was necessary to understand and identify pre-existing site conditions and features that may have affected the pipe material selection. Based on available as-built data and the site walk, it was determined that the existing main was located almost entirely within a four-lane, undivided County roadway, with residential and commercial properties lining the right-of-way on both sides of the road. The site walk also revealed that there were several existing, buried utilities located within the right-of-way and adjacent to the existing water main; existing utilities included a 12-inch water distribution main, a 12-inch gas main, and an 8-inch sanitary sewer line. Based on those preliminary investigations, an initial alignment assessment was developed. The assessment indicated that the new main would have to be located within the roadway and at that same-trench installation would likely be required for at least 50-80% of the new main's alignment.

DETERMINATION OF EVALUATION CRITERIA AND DESIGN REQUIREMENTS FOR MATERIALS ANALYSIS

For pipelines up to 54-inches in diameter, WSSC's Design Standards require the use ductile iron. For pipelines larger than 54-inches, however, their Standards allow for consideration to be given to alternate pipe materials through completion of a pipe materials analysis for the specific project/application. Therefore, it was determined that completion of a pipe materials analysis for the 60-inch replacement main was appropriate, given the function and criticality of the existing main and corresponding need for the its reliability.

Following the initial design tasks, a workshop was held in order to develop an initial set of evaluation criteria on which the pipe materials analysis would largely be based on. The selected criteria included the following:

- Constructability
- Maintenance
- Cathodic Protection
- Schedule
- Cost

While not specifically considered as evaluation criteria, several other design requirements were discussed that would or could affect the outcome of the analysis. These requirements included:

- Consideration of only ductile iron and spiral-welded steel pipe in the analysis; PCCP pipe and bar-wrapped concrete pipe (BWCP) were not considered. HDPE pipe was determined to be unsuitable due to the considerable wall thickness that would be required, given the main's diameter and internal pressures.
- WSSC specified that the ductile iron pipe was to be evaluated based on a manufactured wall thickness equivalent to Special Thickness Class 54.
- For procurement and competition reasons, the specified pipe material and wall thickness must be available from at least two manufacturers for each pipe material.
- Due to the anticipated site conditions and the main's importance, WSSC required the replacement main's exterior to be coated with a corrosion-protectant coating. For procurement and bid competition reasons, WSSC desired to identify at least two types of protectant coatings for each material.
- For ductile iron, pipe joints were anticipated to primarily be gasketed, push-on joints, with the use of mechanical joints at the fittings. For welded steel,

however, several different types of joints were available; therefore, a desktop analysis was necessary in order to evaluate the potential joint types and ultimately recommend a specific joint type to WSSC. The material analysis of welded steel pipe would be completed based on the recommended joint type.

INVESTIGATION OF DESIGN REQUIREMENTS

Following the workshop, it was understood that, except for the elimination of plastic and concrete pipe from the materials analysis, the design requirements needed to be investigated and either confirmed or denied prior to proceeding with the analysis. For most of the requirements, if the specific condition could not be met, it was likely that the given pipe material would be excluded from consideration and there would be no need to complete the analysis because there would only be one pipe material to select.

Availability of Ductile Iron Pipe

For pipe up to 54-inches in diameter, ductile iron pipe is currently manufactured in two different and distinct wall thicknesses – Pressure Class (e.g. Pressure Class 300) and Special Thickness Class, or simply "Class" (e.g. Class 52). In general, pipe with "Pressure Class" designations are thinner-walled than pipe with "Special Thickness Class" designations.

However, for pipe with diameters larger than 54-inches, ductile iron is only available as Pressure Class pipe; Special Thickness Class ductile iron is not commercially manufactured. WSSC's Standards specify that all water mains up to 54-inch diameter shall be ductile iron, with a minimum wall thickness requirement of Class 54; wall thicknesses for ductile iron pipelines larger than 54-inch diameter are determined on a project-specific basis.

For this specific project, WSSC stipulated that the 60-inch ductile iron pipe be manufactured with a minimum wall thickness comparable to Class 54; Class 54 wall thickness was approximated to be 1-inch for a 60-inch pipeline. Wall thickness calculations were performed to confirm that Class 54 pipe was suitable for the expected project and site conditions, and the calculations indicated that Class 54 was a conservative wall thickness for the South Adelphi replacement main.

Because Class 54 ductile iron was not commercially available for 60-inch diameter pipe, it was necessary to contact the various ductile iron pipe manufacturers and confirm that at least two of the manufacturers could meet the following condition:

- Manufacture a 60-inch diameter pipe with a 1-inch wall thickness (Class 54) specifically for this Project.

The following manufacturers were researched and/or contacted to confirm the availability of Class 54, 60-inch diameter ductile iron pipe:

- American Pipe
- Atlantic States Pipe
- Clow Pipe
- Griffin Pipe
- McWane Pipe
- U.S. Pipe

Of those six manufacturers, it was determined that only two produced 60-inch diameter pipe – American Pipe and U.S. Pipe. Subsequent discussions with representatives of both manufacturers definitively confirmed the pipe's availability, and both manufacturers prepared and provided written certification that Class 54, 60-inch diameter ductile iron pipe could and would be produced if ductile iron was ultimately selected.

While discussing the availability of Class 54 pipe with the manufacturers' representatives, the type and availability of potential exterior coatings were also reviewed. Both manufacturers indicated that a wide variety of exterior coatings was available and even an exterior tape coating system (preferred by WSSC) in accordance with AWWA C214 was available if desired; American Pipe was capable of self-applying the tape coating system in their manufacturing plant, but U.S. pipe would have to send their pipe to a third-party tape coat applicator.

Availability of Spiral-Welded Steel Pipe

Spiral-welded steel pipe also required confirmation of at least two manufacturers and the availability of external coatings. Welded steel pipe, unlike ductile iron, is more prevalent and common in larger diameters, and is largely available in diameters from 12 inches to 13 feet. Also unlike ductile iron, the wall-thickness of welded steel pipe is determined on a project-by-project basis; there is no "thickness class", "pressure class" or similar thickness designation for welded steel pipe. Instead, welded steel pipe can be manufactured to specific wall thicknesses up to 0.875 inches.

WSSC's Design Standards do not specify a minimum wall thickness for steel pipe; wall thickness is determined on a project-specific basis. Therefore, standard calculations were completed to establish the minimum wall thickness required in accordance with WSSC's Design Standards and under the following conditions:

- Allowable earth and live loads
- Allowable pipe deflection
- Allowable pipe buckling

Based on WSSC's Design Standards and the anticipated field conditions, most notably being a modulus of soil reaction (E') of 400 psi, the maximum allowable pipe deflection was found to be the determining factor for the pipe's wall thickness; the associated wall thickness was calculated to be 0.72 inches.

Discussions with steel pipe representatives for both American Pipe and Northwest Pipe indicated that both manufacturers could produce a 60-inch diameter pipe with a wall thickness of 0.72 inches. Both manufacturers also confirmed that several different types of exterior coatings, including polyurethane, epoxy and tape wrap, were available for the pipe.

Joint Recommendation – Welded Steel

WSSC's Design Standards specified that, unless otherwise approved, continuous butt-welded pipe joints were the only joint type approved for welded steel pipe. Butt-welded joints are arguably the most conservative and strongest type of joint available, but butt-welded joints are also the most labor intensive and are restrictive from a design perspective. Specifically, due to the way the pipe ends are manufactured, butt-welded joints do not allow for significant, if any, adjustments in the field in relation to either the horizontal or vertical alignment, requiring the Contractor to use extreme precision when installing each segment of pipe. Because of concerns regarding both cost and schedule associated with the use of butt-welded joints, a desktop survey was completed to determine the feasibility of using a joint type other than a butt-welded joint (see Figure 2) for the 60-inch replacement main. The desktop survey was comprised of analyses of available design standards for similar, like-sized municipalities and municipal agencies. Results of the survey are included in Table No. 1.

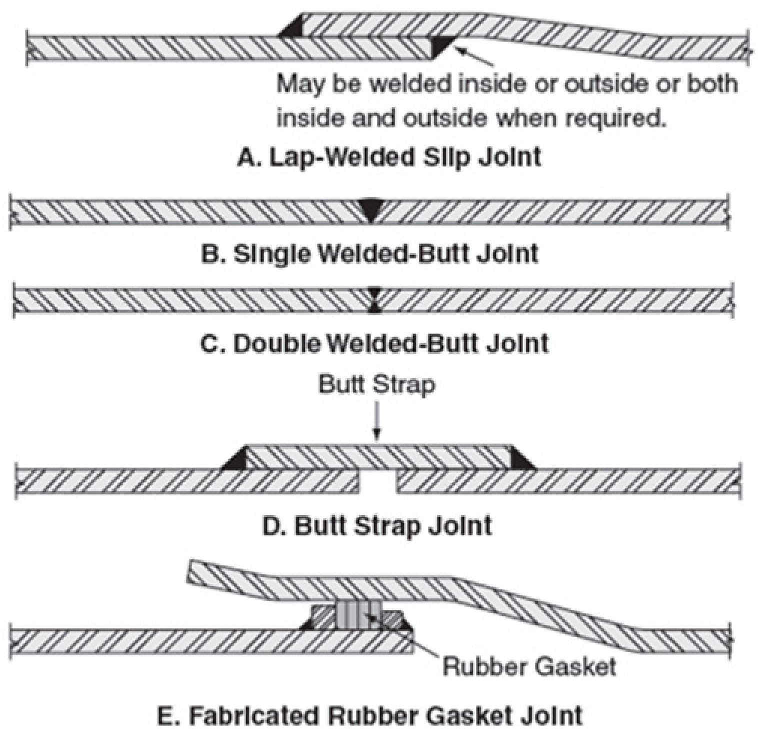

Figure 2. Several Joint Types for Welded Steel Pipe (From AWWA M11, Figure 8-1).

Table 1. Specified Spiral-Welded Steel Pipe Joint Types – Various Municipalities and Municipal Agencies

Municipalities / Authorities	Continuous Butt Weld	Lap Weld (Double)	Lap Weld (Single)	Bell & Spigot (Gasketed)
DC Water	Requires DIP up to 60”			
Boston, MA	Requires DIP up to 60”			
Howard County, MD	X	X	X	X
Baltimore County, MD	X	X	X	X
Phoenix, AZ	X	X	X	X
Denver, CO	X	X	X	X
New York City, NY		X		
San Diego, CA	X	X		
San Antonio, TX	X			
Houston, TX	X			
WSSC	X			

Based on the results of the desktop survey, the following was generalized:

- The use of welded steel pipe appeared to be more predominant in states west of the Mississippi River, especially in those states along the West Coast. Ductile iron was more prevalent along the East Coast, with some municipalities and agencies requiring the sole use of ductile iron for mains up to 60-inches in diameter.
- In instances where welded steel pipe was included in a municipality’s or agency’s design standards, several did not specify a joint type; instead, they required the Engineer to select the joint type during design (e.g. on a project-by-project basis).
- For the surveyed municipalities and agencies that included welded steel pipe in their standards and specified a joint type, all of them required welded joints: continuous butt-welded and/or double lap (fillet) welded joints.

Review of Findings

Following the investigations into the availability of ductile iron and welded steel, a second workshop was held and the findings were presented to WSSC. The findings concluded that both ductile iron and welded steel pipe were available and could be manufactured to meet the project’s design criteria. Also, the various pipe joints available for welded steel pipe were reviewed, and based on the desktop review and consultation with the pipe manufacturers, double lap-welded joints were selected for the project in lieu of continuously butt-welded joints. Lap-welded joints were selected due to the additional flexibility (e.g. joint deflection) available at the joints and expected reductions in construction schedule and cost; double lap-welded joints were expected to result in a shorter construction duration, and therefore a reduction in overall construction costs.

MATERIALS ANALYSIS

From the second workshop, the following pipe characteristics were established to serve as the basis for comparing ductile iron pipe to spiral-welded steel pipe:

Table 2. Pipe Characteristics

	Ductile Iron:	**Welded Steel:**
Wall Thickness	Class 54; approximately 1” thick	0.72” thick
Exterior Coating	AWWA C214 tape coating system	AWWA C214 tape coating system
Joint Type	Push-on and restrained	Double lap-welded
Pipe Length	20 feet	40 feet
Cathodic Protection	Required	Required

Constructability

As indicated previously, there was a strong desire to locate the replacement main in a parallel trench, while leaving the existing main in service, for as much of the alignment as possible. Understanding that, at best, parallel trench installation was only available for about 40% of the alignment, preliminary alignments were developed for both materials to confirm whether one of the materials offered an advantage in terms of more (or less) parallel trench pipe installation. Specifically, welded steel pipe was expected to require a significantly wider trench than ductile iron at the pipe joint locations, approximately 13 feet compared to 8 feet, to allow sufficient room for the exterior joint welds. With that said, the intent was to determine whether this additional trench width for welded steel impacted WSSC’s ability to keep the existing main in service compared to that of ductile iron.

The preliminary alignment designs for both pipe materials indicated that the length of parallel replacement was expected to be about the same and therefore, neither material appeared to offer a distinct advantage in terms of constructability.

Maintenance

Both pipe materials were compared to assess whether one material would be more labor intensive, from a maintenance standpoint. Both mains were expected to have very similar cathodic protection systems and the same number of pipeline appurtenances. The only significant differentiator identified, in terms of maintenance, was that ductile iron consisted of gasketed joints while steel had fully welded joints. Based on the joint type (welded versus gasketed joints) and the number of joints as a function of the expected length of individual pipe segments for each pipe material (40-foot versus 20-foot), a welded steel water main was expected to have fewer maintenance-related issues at the pipe joints over the main's lifecycle than that of ductile iron.

Cathodic Protection

Each main was expected to have full cathodic protection. This consisted of a bonded tape coating (three layer, 80 mil-thickness), bonded joints, and anodes. This is a standard cathodic protection system for steel pipe. Tape coating has had limited use by the ductile iron industry, and only one of the two manufacturers of this size pipe had in-factory taping capabilities. As for bonded joints, the steel pipe would be welded, and the ductile iron pipe would use jumper wires cadwelded to the pipe.

In both instances (tape coating and bonded joints), it was felt that steel pipe was more advantageous than ductile iron. The steel industry possessed the experience for the tape wrap and a welded steel joint was more likely to maintain continuity than a jumper wire on ductile iron pipe.

Estimated Construction Schedule

Construction durations were estimated for both pipe materials (Table 3). Pipe installation was expected to be a slow process for both materials, given the size and expected location of the existing main. With that said, installation of the welded steel main was expected to be more labor intensive because of the welded joints, even though there would be fewer overall pipe joints with the steel pipeline than with ductile iron.

Table 3. Construction Duration

Ductile Iron	22 months
Welded Steel	25 months

Estimated Construction Cost

Preliminary (e.g. 30%) construction costs (Table 4) were estimated for both materials, based on quotes received from the three pipe manufacturers – U.S. Pipe, American Pipe, and Northwest Pipe. In general terms, the material cost of the ductile iron pipe was significantly higher than that of the steel pipe; this was primarily due to the project-specific wall thickness requirement of 1-inch for ductile iron. However, the increased labor costs anticipated for the steel pipe largely offset the difference estimated between the pipes' material costs, resulting in a negligible total cost difference between the two materials.

Table 4. Construction Cost

	Material	Labor	Equipment	TOTAL
Ductile Iron	$8-10 Million	$3-4 Million	$2-3 Million	**$15-$18 Million**
Welded Steel	$6-9 Million	$4-5 Million	$3-4 Million	**$14-$17 Million**

CONCLUSION

Once completed, a third and final workshop was held to review the results of the materials analysis and collectively select a pipe material for the replacement main; the "losing" material was not to be included in the project's design as a "Bid Alternate" or a "Bid Additive". Ultimately, the steel pipe offered a comparative advantage in the areas of maintenance and cathodic protection. While ductile iron was expected to shorten the construction duration, the savings were not considered to be significant and were not expected to result in a significant cost savings compared to the steel pipe. Therefore, welded steel was selected as the pipe material for the replacement 60-inch transmission main.

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C303—A Pipe Material in Search of a History and Searching for a Name

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Abstract
AWWA C303 pipe has been known as Pretensioned Concrete Cylinder Pipe and Bar Wrapped Concrete Cylinder Pipe, without having made any changes to its structure. While this can be confusing, it relates to the hybrid nature of the product. While it has been used extensively in some of the western states and in most of the far western states (those west of the Rockies), it is a new product to the eastern market, where it is currently being promoted. This paper will address the complexity of the nature of the product, relate to its history and provide an insight to its manufacture and design. Although not a new material by any sense of the word, it is new to many potential users; owners, engineers and contractors. Also, the eastern market will almost certainly involve some variations to the product's historical experience. These variations should not cause undue concern as long as they are recognized. History, experience and knowledge are essential ingredients in providing a beneficial new tool in the pipe industry's tool box.

Background

C303 pipe is somewhat of an enigma. The title presently is Concrete Pressure Pipe, Bar-Wrapped, Steel Cylinder Type, but it may not contain any concrete, only mortar. While all other AWWA concrete pipe designations (AWWA C300 Reinforced Concrete Pressure Pipe, Steel-Cylinder Type; C301 Prestressed Pressure Pipe, Steel-Cylinder Type; C302 Reinforced Concrete Pressure Pipe, Non-Cylinder Type – henceforth denoted by their AWWA designation) are characterized by a rigid pipe design, C303 is designed as a flexible conduit. While C300, C301 and C302 pipe have generally been limited to 16 and 20 foot pipe lengths (although C301 has been produced in 24 foot lengths), C303 is manufactured in lengths from 24 feet to 40 feet; primarily from 32 to 40 feet. Where C300, C301 and C302 are not restricted as to their maximum diameter, C303 has been restricted.

To get a full measure of C303 pipe, we must compare it to Shot Cote Pipe (S/C), Modified Prestressed Pipe (MPP), Concrete Cylinder Pipe (CCP), P-303 Pipe and Pretensioned Concrete Cylinder Pipe (PCCP). While this might appear to be a daunting effort, it is really quite simple. These are all the same product. They are names that have attached themselves to the product at different times, or different companies, or different locations in the product's life.

Even AWWA C303 has experienced several name changes, primarily interchanging "pretensioned" and "bar-wrapped" as can be seen in the following section. How important is this? As Shakespeare wrote, "That which we call a rose by any other name", etc. Personally, we like the way old Ben Franklin put it, "What signifies knowing the names, if you know not the nature of things".

While the name has changed, the essential components, design and manufacture of the product have remained the same.

History

Figure 1 – C303 Pipe

Pretensioned Concrete Cylinder Pipe was introduced by American Pipe and Construction Company, a predecessor to Ameron, in 1942. Today, there is interest regarding the original criteria governing the design. However, the reasons for the distribution of the steel rod to cylinder ratio, established in those early days of development, are lost to 73 years of history, as is the criteria established for the minimum cylinder gage for the various diameters. An argument can be made that the cylinder was established to enable it to resist deformation during the placement of the lining in the long lengths allowed by the C303 criteria. The rod to cylinder distribution could logically be based on an allowable spacing of the rod and the ability, at that time, of drawing and bending the heavier rods; thus establishing a limitation on the maximum area of steel for each diameter pipe. If there was any consideration given to the additional corrosion resistance that a heavier gage cylinder might impart to the water tightness of the product, it should be remembered that there was limited knowledge of the corrosion process in 1942 and the thinking at the time was that a high alkalinity cement mortar naturally protected encased steel. Whatever the reasons, over time, the company migrated to the cylinder controlling the design by requiring a minimum of 60% of the total required steel in the cylinder, with the remainder in the rod wrap. Based on current published literature, Ameron presently maintains this distribution.

The first AWWA C303 standard was approved in 1970, although the product had been used extensively in the western and southwestern sections of the United States for many years prior to that time. This first AWWA standard was based on the requirements for C303 pipe contained in the Federal Specification SS-P-381a

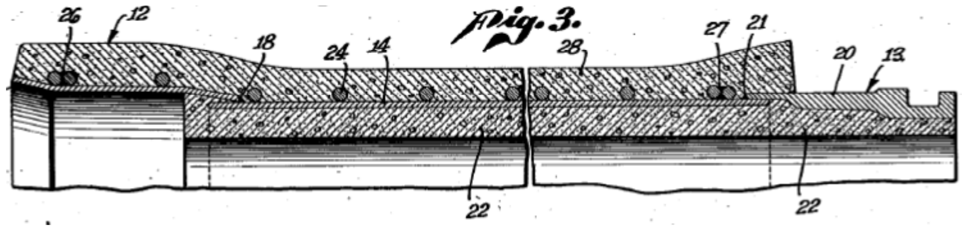
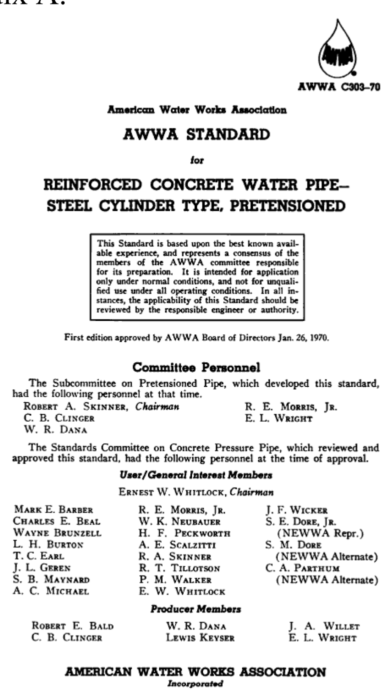


Figure 2, Original Patent Drawing

(originally designated SS-P-000381 in 1953) that had been used by the industry for approximately 17 years. The pipe was titled Reinforced Concrete Water Pipe-Steel Cylinder Type, Pretensioned. Diameters were limited to 10 inch to 42 inch. While the pipe was classified as rigid for smaller diameters (up to 21 inch) and semi-rigid for the intermediate and larger diameters, it was acknowledged in the foreword that "generally, the pipe cannot economically be designed as a structural member having flexural capability to support, without dependence on lateral restraint provided by passive earth pressure, the external loads imposed on pipe buried underground". The minimum cylinder thickness ranged from 16 gage for 10 inch pipe to 12 gage for 42 inch pipe. Similar to the requirement in the Federal Specification, the steel cylinder was required to have a minimum of 40% of the total steel requirement. The design procedure was established in Appendix A of the standard.

The first revision to C303 was in 1978. It was titled Reinforced Concrete Pressure Pipe, Steel Cylinder Type, Pretensioned, for Water and Other Liquids; for obvious reasons. Among a few other minor modifications, it decreased the minimum allowable cylinder thickness to 18 gage in the smaller diameters. Pipe design was again established in Appendix A.



AWWA C303-70

American Water Works Association

AWWA STANDARD

for

REINFORCED CONCRETE WATER PIPE— STEEL CYLINDER TYPE, PRETENSIONED

This Standard is based upon the best known available experience, and represents a consensus of the members of the AWWA committee responsible for its preparation. It is intended for application only under normal conditions, and not for unqualified use under all operating conditions. In all instances, the applicability of this Standard should be reviewed by the responsible engineer or authority.

First edition approved by AWWA Board of Directors Jan. 26, 1970.

Committee Personnel

The Subcommittee on Pretensioned Pipe, which developed this standard, had the following personnel at that time.

Robert A. Skinner, *Chairman*
C. B. Clinger
W. R. Dana
R. E. Morris, Jr.
E. L. Wright

The Standards Committee on Concrete Pressure Pipe, which reviewed and approved this standard, had the following personnel at the time of approval.

User/General Interest Members

Ernest W. Whitlock, *Chairman*

Mark E. Barber
Charles E. Beal
Wayne Brunzell
L. H. Burton
T. C. Earl
J. L. Geren
S. B. Maynard
A. C. Michael
R. E. Morris, Jr.
W. K. Neubauer
H. F. Peckworth
A. E. Scalzitti
R. A. Skinner
R. T. Tillotson
P. M. Walker
E. W. Whitlock
J. F. Wicker
S. E. Dore, Jr. (NEWWA Repr.)
S. M. Dore (NEWWA Alternate)
C. A. Parthum (NEWWA Alternate)

Producer Members

Robert E. Bald
C. B. Clinger
W. R. Dana
Lewis Keyser
J. A. Willet
E. L. Wright

AMERICAN WATER WORKS ASSOCIATION
Incorporated
2 Park Avenue, New York, N.Y. 10016

Figure 3 – First AWWA C303

The next revision in 1987 retained the 1978 title and allowed a maximum diameter of 54 inches. The minimum allowable cylinder gage was changed back to 16 in the smaller diameters. While it recognized AWWA Manual M9, Concrete Pressure Pipe as a supplement to the standard, the revision maintained the Appendix A design.

The 1995 revision changed the name of the product to its current variant, Concrete Pressure Pipe, Bar-Wrapped, Steel-Cylinder Type. Major revisions included the increase in allowable diameter to 60 inches and the increase in the allowable stress in the steel members for working and surge pressures from 16,500 psi and 24,750 psi to 18,000 psi and 27,000 psi respectively. Reference to M9 for design replaced Appendix A.

In 2002, the revision again increased the allowable diameter; this time to 72 inches. The 2008 revision, the current edition, had no substantive changes.

An excellent paper (Bardakjian, Murphy, 2013) regarding the early history of this product and the development of its structural concepts can be found in the 2013 ASCE Pipeline Division Conference Proceedings.

Manufacture

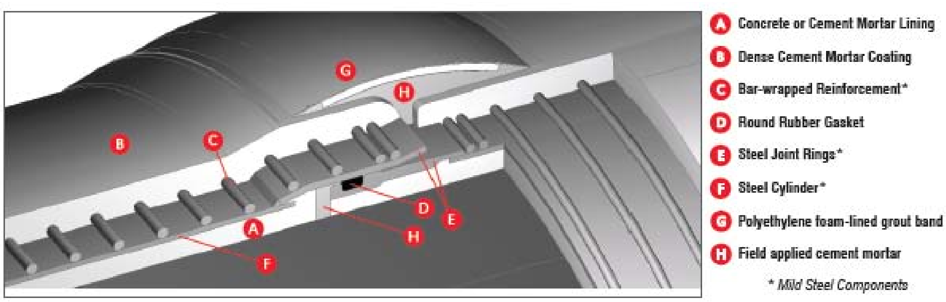


Figure 4 – Typical C303 Pipe Joint Detail Cut Away

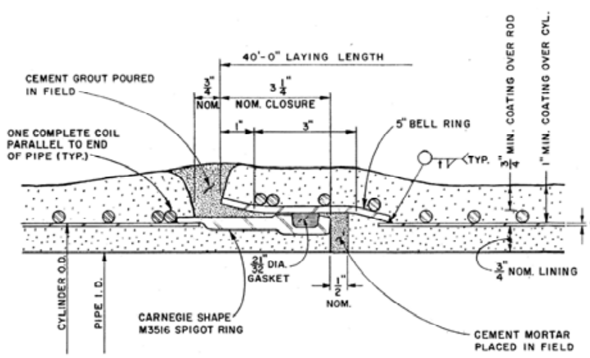


Figure 5 – Typical C303 Pipe Joint Cross Section

C303 manufacture begins with the rolling of a relatively light gage cylinder to the appropriate external diameter to allow for the placement of ½ inch mortar or concrete lining for 16 inch and smaller pipe and ¾ inch for larger diameters. The cylinder is generally formed by the spiral welding method. A Carnegie spigot and expanded bell joint rings are welded to the ends of the cylinder to provide for gasketed field joining of the pipe. The individual cylinders with joint rings are then hydrostatically tested to a pressure to induce a stress equal to 75 percent of the minimum specified yield strength of the steel cylinder.

Figure 6 – Shop Hydrostatic Testing

Next a mixture of cement mortar or concrete is centrifugally cast on the inside of the cylinder to the required thickness, from the face of the spigot to a point allowing full engagement and joint mortaring on the bell. The lining is then steam cured for 6 hours or water cured for 24 hours, minimum. The cement-mortar (or concrete lining) is required to achieve a 28-day compressive strength of 4,500 psi.

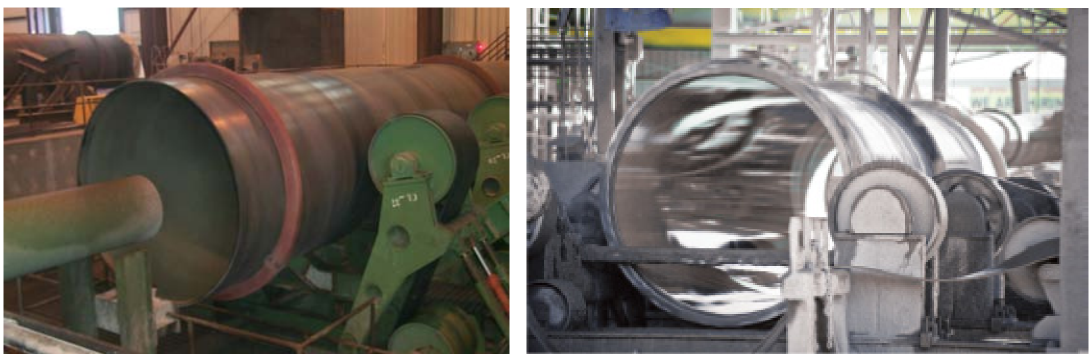


Figure 7 – Cement Mortar Lining

Following curing, the cylinder is circumferentially wrapped helically with a continuous reinforcing bar at a stress of between 8,000 and 10,000 psi. Pulling the bar around the cylinder at this stress ensures conformity of the bar to the outside of the steel cylinder. As the bar is wrapped around the cylinder, Portland-cement slurry is applied to the cylinder, just ahead of the bar, so that there is a slurry coating between the cylinder and bar surfaces. The bar is anchored by welding to the joint rings, not to the cylinder. See Figure 8.

The pipe then receives a Portland-cement slurry simultaneously with the cement-mortar coating to establish a high alkalinity environment to the surface of the steel bar and cylinder. The coating is then steam cured for a minimum of 12 hours prior to shipping. See Figure 9.

Figure 8 – Bar Wrapping

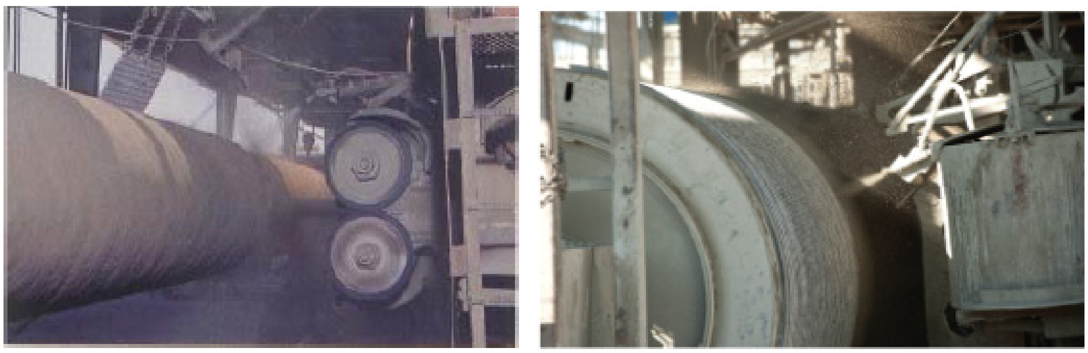


Figure 9 – Cement Mortar Coating

With the exception of beveled pipe ends for minor deflections, as well as outlets in the barrel of the pipe, more complex fittings are produced much like cement mortar lined and cement mortar coated steel pipe, with heavier cylinders than the pipe barrel and no bar wrapping. Two piece elbows are frequently fabricated from finished pipe, requiring a wrapper plate at the location of the elbow seam.

Figure 10 – Typical Fittings

Design

Internal pressures are treated in the same manner as other flexible pipeline products. The cylinder and bar wrapping provide the complete mechanism for resisting the internal pressures. However, as the bar does not present a continuous lateral structural component to the pipe, the T in the Barlow Equation, T = PD/2S is

substituted with A, the area of steel per foot of pipe, by multiplying both sides of the equation by 12, changing the formula to: A = 6PD/S
where:

P = internal pressure, psi
T = steel cylinder thickness, in.
S = Allowable stress, psi
D = inside diameter of the steel cylinder, in.
A = area of steel for cylinder and bar, in.^{2}/foot of pipe

For the analysis of the external load carrying capacity of the pipe, the Modified Iowa Formula is utilized,

$$\Delta x = \frac{D_l k(W/12)r^3}{(EI + 0.061E'r^3)}$$

Where:

Δx = horizontal deflection of the pipe, in.
D_l = deflection lag factor
k = bedding constant
W = load per unit of pipe length, lb./lin. ft.
r = radius of pipe, in.
EI = pipe wall stiffness, in^4/lin. In.
E' = modulus of soil reaction, psi

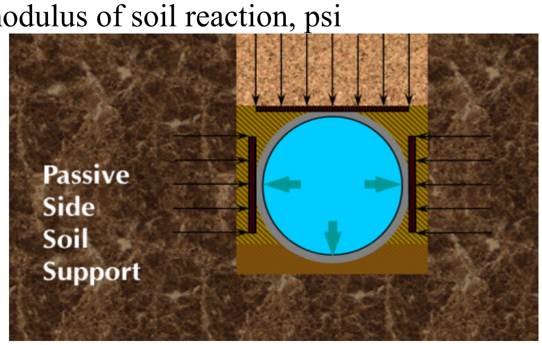


Figure 11 – Flexible Pipe Side Support

Unlike its cousin, cement mortar lined and cement mortar coated steel pipe, which can exhibit a similar external appearance, and one in which the EI is calculated by adding the transverse moment of inertia of the individual rings, C303 is analyzed as a composite section. This is due to the fact that the bar-wrapping imparts some compression in the interior lining and the steel bar acts as a “key” to better bond the exterior coating to the steel cylinder. E is taken as the modulus of elasticity of the concrete or cement mortar (4,000,000 psi). I, the transverse moment of inertia of the composite wall section, is developed by transforming the steel components to concrete and analyzing the complete wall cross section at the center of gravity. Per M9, I is limited to 25 percent of its computed value in determining the value of EI applied to the Modified Iowa formula. This composite EI reduction is based on the possibility of minor cracking (0.01-inch) developing in the cement mortar elements at the maximum allowable deflection.

For the analysis of the external load resistance, the allowable deflection of the C303 pipe per M9 is $D^2/4000$. This value must be equal to, or greater than, the value of the calculated Δx. By establishing the deflection lag factor, D_l equal to 1.0, a reasonable value for a pressurized pipe, and calculating EI based on the steel required to resist the internal pressure, the allowable load can be computed by:

$$W = \frac{D^2(EI + 0.061E'r^3)}{333kr^3}$$

If the height of cover over the top of the pipe is less than 8 feet, then an AASHTO HS-20 live load must be incorporated into W as well as the earth load.

Installation

While C303 pipe can be manufactured in diameters ranging from 10 inches to 72 inches, it would be expected that the eastern market would only see the transmission main diameters; those allowing manned entry to complete the interior joints.

As with any installation, the contractor starts with a safe trench that has been dewatered. A layer of compressible bedding material should be placed in the bottom 2 to 6 inches of the trench to prevent the pipe from resting on any hard objects or an unyielding foundation. This material should not be manually compacted. Its purpose is to allow the pipe to settle into the bedding to achieve a cushioning effect.

The pipe is generally installed with the bell end facing the direction of laying. Much like prestressed concrete cylinder pipe and gasketed steel pipe, the gasket is stretched around the spigot end and tension relieved. The gasket and the existing bell end should be liberally coated with an approved lubricant. The spigot is inserted in the bell end with the pipe being installed parallel to the pipe previously laid and the pipe pushed home. The force to accomplish the joining is generally created by the backhoe and the use of a choker sling. Following insertion, a feeler gage should be used on the outside of the joint to assure that the gasket is in its proper place.

Prior to backfilling the pipe, the exterior joint gap is diapered and filled with a high slump cement mortar mixture. This should be poured from one side of the pipe and allowed to rise on the other side, assuring good distribution under the pipe and at the haunches. See Figure 12.

Because the effect of EI for C303 pipe provides a much more rigid pipe structure than other flexible pipe products, C303 is frequently categorized as a semi-rigid pipe, which by its nature, also limits the allowable deflections to much less than that allowed for other flexible pipe products (ductile iron pipe limits deflection to 3 to 5%, depending on the lining; steel pipe limits deflection to 2 – 5%, depending on the lining and coating systems; plastic pipe materials may allow up to 7.5%). In the very small diameters, it could be categorized as rigid pipe (but still designed using flexible pipe theory). However, as previously noted in the forward to AWWA C303, the soil plays a major role in the resistance of the pipe to the external load. As the diameter increases, the role of the soil greatly increases in importance. Placement of the properly compacted soil envelope up to a minimum of seven tenths of the height of the pipe is very important. The compactive effort should be compatible with the soil

Figure 12 – Exterior Joint Grout

material and consistent with the E' selected for design. Extra care should be taken to assure good compaction and support in the pipe haunch area.

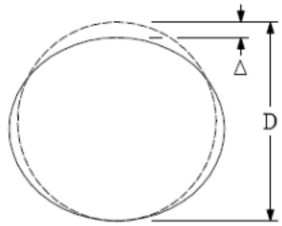


Figure 13 – Deflection

The backfill from this point to a height of one foot above the top of the pipe should be free from large rocks or boulders and should be compacted to a degree required for the support of the desired grade. Figure 14 depicts pipe backfilling operations.

After being backfilled, the pipe should be entered and the interior of the joint pointed up with a two parts sand to one part cement mix, stiff enough to be troweled and remain in place. An approved adhesive material placed around the upper hemisphere of the joint may assist in this operation. See Figure 15.

Restrained joint C303 may require a fully circumferential inside or outside weld. Naturally, this operation would precede the backfill operation if an outside weld is required and precede the interior joint pointing operation if an interior weld is required. All welding should be done by certified welders; either certified to AWS or ASME standards.

As with any pipe, the open end(s) should be plugged or capped each night to prevent water, mud, or critters from getting into the pipe.

East Coast Issues

While C303 pipe has experienced an excellent 73 year history, this history was essentially exclusively in the western states. Conditions in the eastern states are frequently quite different. Heavy population concentrations, a lack of open fields, corrosive clay soils and higher water tables are some of the conditions generally found for an eastern pipeline. (See Figure 16.) Shoring boxes required for trench

Figure 14 – Backfilling Large and Small Diameter Flexible Pipe

Figure 15 – Interior Joint Grout

safety are more prevalent in the east. These conditions frequently require a limited pipe section length. While all of these issues can be overcome, they can also change the economics associated with the pipe.

Geography is another potential impediment. In the west, Ameron operated 6 or 7 manufacturing plants in California, one each in Phoenix, AZ, Albuquerque, NM and Portland, OR. United Concrete Pipe Company, now defunct, operated plants in Baldwin Park and Riverside, CA, Pleasant Grove, UT, Aurora, CO, and Dallas, TX. Gifford Hill American, now Hanson, operates 3 plants in Texas. Hanson has relatively recently retrofitted Palatka, FL, to manufacture C303, but that is the only location east of the Mississippi in such operation.

However, these are explained simply to recognize several of the competitive issues.

This by no means should enter the thinking of the designer or owner when considering the product on its technical merits for inclusion in their project.

Recommendations

Cylinder thickness: C303 pipe, like all products, must be correctly designed, manufactured, installed and operated. The AWWA requirements for the pipe has

Figure 16 – High Water Table and Trench Box

morphed from 42 inch maximum diameter to 72 inch maximum diameter along with allowing for higher stresses in the steel cylinder, which in turn yields higher strains in the exterior cement mortar coating. Additionally, as the pipe diameter increases, C303 pipe reacts more similar to flexible pipe than semi-rigid or rigid pipe, therefore relying more on the soil envelope embedment rather than the pipe stiffness. We believe that the steel distribution of 60% of the total steel area being required in the cylinder better serves the owner for a longer service life for the pipe. The 60% cylinder requirement adds no steel to the product as the design is efficiently controlled by the internal pressure and the modulus of soil reaction, E', and not the EI stiffness function in the Modified Iowa Equation. As the cylinder is the sole component providing water tightness, this minimum level provides added protection from corrosion penetrating the watertight membrane. The cylinder is also the component providing axial strength for the pipe. The axial stress due to a full thrust condition is one half (0.5) of the hoop stress at any given pressure (Luka, Ruchti, 2008). Therefore, a pipe having at least 50% of the required steel in the cylinder, allows the pipe to resist any present or future relocation full thrust condition.

Inspection: Upon commencement of manufacturing for a project, the facility should be visited to assure that all proper procedures and testing are being accomplished. It is generally beneficial if the visit includes any first time users of this pipe, be it designer, owner or contractor. Understanding the nature of a material benefits all involved.

Figure 17 – Plant Inspection

Soil Compaction: Soil compaction requirements are also important. Many times, an 85% standard proctor density per ASTM D698 is satisfactory to obtain the necessary support for the pipeline. The use of the spoil material for bedding and backfill can reduce the cost of the installation substantially. However, a soils engineer should review the material to insure that it meets the criteria for the E' as required by the pipe design. It is usually wise to specify a higher level of compaction than might be otherwise necessary to assure the owner is getting the compaction actually required. If the contractor is achieving good consistent results, it is always possible to be a bit less stringent in the field.

Jetting and vibrating the soil pipeline envelope is fairly common in the western states. However, this process should never be attempted in a cohesive soil. The material should possess a good, free-draining ability before a jetting compaction operation should be allowed.

Figure 18 – Soil Compaction Testing

Finally, any contractor installing this pipe for the first time should be made fully aware of the flexible nature of the product. While C303 pipe has greater rigidity than many other flexible pipe materials, it is not the rigid concrete pressure pipe to which owners and contractors in the eastern states have become familiar.

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AWWA Manual M9 Concrete Pressure Pipe
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Exploring Use of Large-Diameter HDPE Pipe for Water Main Applications

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Abstract

Three workshops were held as part of a Water Research Foundation (WaterRF) research project to investigate durability and reliability of large diameter HDPE pipe (WaterRF Project #4485). As stated in this paper, the use of large diameter HDPE pipe for water applications is currently very limited, as most water utilities are not fully familiar with characteristics and capabilities of this type of pipe. The objectives of these workshops were to explore issues and concerns about use of HDPE pipe in large diameter transmission mains. The workshop participants were from water utilities, design consultants, project team members, and pipe manufacturers and installers. Through innovative brain storming sessions, the issues and concerns regarding use of large diameter HDPE were identified and subsequently ranked. This paper provides details of these workshops, topics discussed, concerns and issues with use of HDPE pipe, and strategies to resolve these concerns and issues. As results of these workshops, a set of strategies for water utilities, design consultants, and pipe manufacturers were developed and presented in this paper.

INTRODUCTION

The Report Card on America's Drinking Water Infrastructure, states that U.S. infrastructure is in poor condition, and in the coming decades, the cost of renewing water infrastructure could reach more than $1 trillion (ASCE, 2013). Approximately 33 percent of drinking water is lost each year (Radoszewski, 2009). Due to leaks and breaks, water utilities in the United States lose 40 billion liters (10.6 billion gallons) of water between treatment plants and tap everyday out of 160 billion liters (42.3 billion gallons) of processed water. The U.S. spent approximately $1.2 billion on water pipe rehabilitation in 2006 when the need was to spend $6 billion (Najafi, 2005). In a study by the Plastic Pipe Institute (2009), it was found that while HDPE pipes have been used for municipal water applications for almost fifty years, still they are minimally used for potable water transmissions/distributions, and wastewater services.

An element of the research project to investigate durability and reliability of large diameter HDPE pipe (Water Research Foundation Project #4485) called for holding project workshops with industry professionals to seek input on the critical issues to be addressed during the course of this project. To fulfill this requirement, three workshops were organized. This paper provides the highlights and findings of these workshops. The objectives of the Project Workshops were to obtain as much input as possible from the participating industry professionals from water utilities, HDPE manufacturers/vendors and Plastics Pipe Institute (PPI) representatives by conducting small and large group discussions.

The workshops were held in conjunction with industry events (PPI Municipal Board Meeting, April 2013; ACE13; and ASCE Pipelines 2013) to maximize participation and minimize travel costs. Potential participants were invited through e-mail invitation. Delphi technique, brainstorming technique, and breakout sessions were among the strategies utilized to maximize participation from the attendees.

The following topics were covered during the workshops:

- What Constitutes Large Diameter HDPE Pipe
- Identification of Critical Issues
- Discussion of High Priority Topics

A summary of the discussions regarding these topics is provided in the subsequent sections of this paper.

WORKSHOP DETAILS

Discussion of what constitutes large diameter HDPE pipe

The original project scope had identified 24 inches as the boundary for categorizing HDPE pipe as "large diameter." A discussion of this issue during workshop #1 revealed that almost all of the participants and specifically the utility representatives participating in workshop #1 felt that the threshold for large size HDPE pipe is 16 inches. The participants indicated that the lowering of the size threshold will expand the experience base with use of HDPE, as history of use with larger pipe sizes may not be extensive. As a result, the project team with concurrence from the WaterRF Project Manager decided to lower the threshold from 24 inches to 16 inches.

Identification of critical issues

During the brainstorming session of workshop #1, the participants offered various issues that could be of critical significance to understanding the durability and reliability of HDPE pipe. Overall, 22 issues were identified during workshop #1. Additional issues were offered by participants of workshop #3. Table 1 summarizes the issues raised by the participants of the three workshops with more details provided in the following sections.

Table 1. Issues raised by workshop participants.

Issue	Workshop	Title
1	1	Perception Issue
2	1	Third Party Damage (Outside Damage)
3	1	Comparison to Other Pipe Products
4	1	Installation Aspects/Contractor
5	1	Proven Track Record – EUROPE
6	1	Modes of Failure
7	1	Amount of Maintenance – Life Cycle Cost Analysis
8	1	Service Life
9	1	Life Reliability Curves
10	1	Specifications, Design, Installation/Contractor, Inspection, & Maintenance
11	1	Asset Management Plan
12	1	Connection/Fittings
13	1	PE Material History/Variations
14	1	Permeations of Hydrocarbons
15	1	Disinfection Byproducts Impact
16	1	Seismic Activities
17	1	Regional Issues
18	1	Freeze/Thaw
19	1	Expansion/Contraction – Effects on Fittings
20	1	Trenchless Installation – Scoring
21	1	Jointing Methods/Fusion, Mechanical
22	1	Fusion at Colder Temperatures
23	3	Change of Surface Conditions
24	3	QA/QC of Manufacturers
25	3	Life Cycle Cost
26	3	Internal Abrasion
27	3	Lifetime Prediction Curve
28	3	Training/Qualifications

Issue	Workshop	Title
29	3	Supply Chain Management
30	3	Tracking (Asset Management)
31	3	Learning from other Applications (Example: Book on use of HDPE for Ocean Outfalls)
32	3	Time to Repair & How to Repair
33	3	Lead-time for Fittings

Following the brainstorming sessions, the participants were asked to rank the issues for further discussion. Figure 1 shows the total scoring provided by participants based on workshop leader instructions for each of the 22 issues during workshop #1.

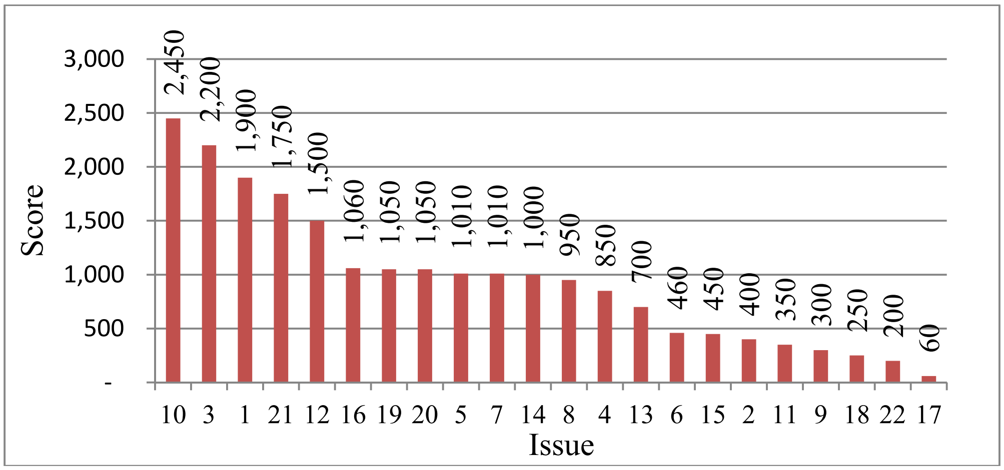

Figure 1. Ranking of issues by workshop #1 participants.

The scoring chart showed a clear delineation between issues 10, 3, 1, 21 and 12 on the one hand and the remaining issues on the other. Table 2 shows the list of these top five issues picked by the participants of workshop #1 for further discussion. Three breakout groups were formed to discuss these five issues.

Table 2. Top five issues from workshop #1.

Issue	Top Five Issues	Group
10	Specifications, Design, Installation/Contractor, Inspection, Maintenance	1
1 & 3	Perception Issue & Comparison to Other Pipe Products	2
21 & 12	Jointing Methods/Fusion, Mechanical, Connection/Fittings	3

Workshop #2 participants selected eight issues as listed in Table 3 as high priority. When asked to limit the prioritized issues to five topics only, the participants essentially selected the same issues as the participants of workshop #1, further validating the critical nature of these issues. The five issues highlighted in bold in Table 3 were discussed in further detail during workshop #3.

Table 3. Short-listed issues from workshop #2.

Issues	Topics
1*	**Design**
2	**Installation**
3	**Repair and Operations & Maintenance (O&M)**
4	Change of Surface Conditions
5	QA/QC of Manufacturers
6	Life Cycle Cost
7	**Perception**
8	**Connections/Fittings**

*The highlighted items were discussed in more details at Workshop #2

During workshop #3, the participants added 12 issues to the list of issues as shown in Table 1. An open discussion was held and the participants offered their perspectives and concerns. Lively discussion of issues of interest to participants took place during workshop #3.

Discussion of High Priority Topics

During the course of each workshop, detailed discussions of high priority issues were conducted. During workshop #1, the participants were divided into three small groups to discuss high priority issues. However, for workshop #2 and #3, a project team member engaged all the participants in a discussion of high priority issues. The following topics were discussed in detail:

Perception Issue

The participants of workshop #1 identified perception as a high priority issue. The participants of workshop #2 and #3 also concurred with this characterization. The participants felt that the utility engineers and engineering consultants generally perceive HDPE pipe as being suitable for small diameter and/or low pressure applications, and as a result automatically rule it out as an option for large diameter pressure pipe applications.

Table 4 summarizes the reasons behind the perception issue based on comments provided by workshop participants.

Table 4. Reasons for perception issues.

Broad Reason	Specific Reasons
Lack of knowledge about the product	1. In order for utility to approve the use of HDPE for large diameter pressure application, there is a need to have acceptance from all the stakeholders in the utility including decision makers, specification writers, field staff and users. This would require a high level of education and engagement. 2. Two big issues are training and familiarity. Utility workers want to be comfortable with using a product and familiar with the repair methods and materials. 3. Perception that HDPE is not for water application

Perceived risk associated with the use of a material a utility has not used in the past	4. HDPE is a new product for this type of application 5. HDPE is not in our comfort zone; we do not have experience with it. 6. Some associate HDPE with Polybutylene pipe which has had a negative history 7. Utilities are resistant to change. They need a driver to change. The perception is that since the utility has not used HDPE for large diameter pressure application in the past, there may be unknown risks associated with its use.
Other	8. Requires new tools and equipment 9. Requires additional inventory items for repair 10. Cost is a consideration

The workshop participants also offered a number of strategies to overcome the perception issue as listed in Table 5.

Table 5. Strategies to overcome perception issues.

Strategy	Description
1	Success stories and lessons learned – take advantage of the experiences of utilities that are using HDPE and share their stories. Failures can also be a great learning tool.
2	Need to hear testimonials from utilities. These will resonate with other utilities.
3	Establish a Center of Excellence for HDPE Pipe to promote "Best Practices" for HDPE pipe.
4	Highlight the advantages of HDPE pipe such as its leak free nature due to butt-fused joints.
5	Utilities that provide both water and gas service can be more inclined to use HDPE for water applications as they already may have an experience base with use of HDPE for gas applications.
6	Education is the key. Must educate staff so that they are familiar with the material, installation and repair methods, etc. As an example, many utilities are willing to use HDPE for complex, environmentally sensitive projects that typically involve trenchless installation by horizontal directional drilling (HDD) or pipe bursting. However, the same utilities do not consider HDPE suitable for less complicated projects. Education can help utilities overcome this dichotomy.
7	Life cycle cost – too much emphasis is often placed on the pipe cost and not the bigger picture. Must factor into decision the life of the pipe, maintenance costs etc. to get the full picture. As an example, in Colorado Springs, material price for HDPE is higher than ductile iron but there are other considerations including HDPE response to dynamic pressure, soil conditions and seismic activity. HDPE can become more cost competitive for large diameter applications when life cycle costs are considered.

8	Highlight the specific applications for HDPE. Identify usage in right applications. Help utilities understand where it makes sense to use. As an example, Colorado Springs indicated they have had failures and growing pains. Their drive to use HDPE started with corrosion issues.
9	Contractors have a lot to offer and can be helpful, need to listen to their experiences.

Design, Installation/Contractor, Inspection & Maintenance Issues

The workshops participants overwhelmingly expressed an opinion that comprehensive specifications, along with accurate design, proper installation, and timely maintenance would offer a long lasting solution for a pipeline project, regardless of the pipe material used. The participants identified a number of needs related to these issues as listed in Table 6.

Table 6. Design, installation/contractor, inspection and maintenance issues.

Issue	Details
Design	There is a need for experienced and trained design engineers
Pipe Manufacture	HDPE is offered in many sizes, wall thicknesses and cell classifications. While this versatility provides flexibility, it also can cause confusion.
Tapping & Repair	Procedures for tapping and repair of HDPE as well as how to properly connect to other pipe materials are not readily available. The latter issue is specially impacted for low DR pipes where the thick HDPE pipe may require a reducer to match the outer diameter of the cast iron, ductile iron or PVC pipe it is being connected to.

The workshops participants offered a number of strategies to address the issues related to specification, design, installation, and maintenance as listed in Table 7.

Table 7. Strategies to address design, installation & maintenance issues.

Strategy	Description
1	Industry should consider providing regular training for design engineers
2	Industry should consider developing design tools for engineers to use
3	Utilities should use Quality-based Selection (QBS) process to select qualified design consultants. Selection based on price can lead to inferior design
4	Utilities should consider specifying an acceptable level of qualifications for contractors
5	Contractors should strive to hire trained personnel or offer full training and supervision for their personnel who may not be fully experienced
6	Industry should consider certification at various levels to improve quality
7	Industry should consider developing design, installation, and maintenance guidelines similar to guidelines developed by American gas association (AGA)
8	Industry should consider collecting and compiling specifications developed by various utilities and making it available to all users

Strategy	Description
9	Inspection during production, delivery and installation is critical for long-term success. Inspector training and certification should be considered by the industry.
10	Gas pipeline contractors should be encouraged to consider serving the water market
11	Pipe manufacturers should consider having regular field observations to promote best practices
12	Specification should address all critical issues including requirements for equipment, proof testing, groundwater control, backfill requirements, and acceptance testing requirements
13	Industry should consider developing standard guidelines for maintenance aspects such as repair of HDPE pipe and tapping of HDPE pipe
14	Industry should consider providing training and certification for HDPE pipe repair professionals
15	Industry should consider developing guidelines for non-destructive evaluation of HDPE and provide a recommended schedule for inspection based on a set timetable or based on the bathtub curve
16	Utilities should consider engaging qualified professionals to perform forensic evaluation of failure incidents to learn from the failure and ensure the root cause of failure is established and eliminated from future design. During forensic evaluation, it is critical that the field personnel be interviewed as they are often most knowledgeable about what might have led to the failure.
17	A simplification of HDPE pipe product line items may be beneficial to reduce confusion
19	The consequence of failure should be considered as a factor in pipe material selection. The consequence of failure should be quantified in dollar terms and should consider financial loss due to failure (for example if the water supply to a hotel is interrupted).

It was the strong view of workshops' participants that there is a need for the development of uniform specifications, and guidelines for design, installation and maintenance of HDPE pipe, and the benefits such documents would offer to the utilities that decide to specify HDPE for large diameter pressure applications. While the Plastic Pipe Institute (PPI) and the American Water Works Association (AWWA) have published standards and guidelines for use of HDPE, uniform specifications which utilities can readily use are not available. The participants of workshop #1 developed the following list for the items that should be addressed in specifications for HDPE pipe.

1. Fittings
2. Fusion process requirements
3. Mechanical connections
4. Quality Assurance/Quality Control
5. Testing
6. Certifications
7. Design specifications

   a. Connection to other materials
   b. Joint Restraints
   c. Thermal movement
   d. Poisson effects
   e. Disinfection (Chlorine)
8. Training
9. Inspections (pre and post)
10. Construction specification
   a. Bedding/hunching and backfill
   b. Handling
   c. Trenchless specifications
   d. Fitting specifications
11. Repair methodology
12. Equipment qualification
13. Installer qualification
14. Geotechnical specifications
15. Design life

Jointing Methods/ Fittings (Fusion & mechanical) Issues

The workshops participants frequently brought up the issue of fittings. While butt fusion was considered as an established process for joining pipe sections, there seemed to be a need for a better understanding of options for fittings and connecting of HDPE to other pipe materials. Table 8 summarizes the issues brought up by workshops participants.

Table 8. Joining methods/fitting issues.

Issue	Details
Availability of Fittings for large Diameter HDPE Pipe	1. Not all HDPE pipe suppliers offer HDPE fittings and the utility has to search for other vendors for such fittings. Fittings are only available for smaller pipe sizes.
Information on Joining Methods/Fittings	2. There is a need for procedures to make the fittings, such as MJ and saddle requirements 3. There is a need for a sourcebook on information on fittings and jointing 4. There is a need to know what works and what does not work as far as fittings are concerned 5. There is a need for standard specifications for HDPE and PVC connections
Lack of Training	6. Installation of large diameter applications needs specialized training 7. Contractors without proper and specialty training leads to substandard installations

Issue	Details
Other	8. There are a number of issues with connecting HDPE to other pipe materials which are often referred to as "end-of-the-pipe" problems 9. MJ adapters do not work for connecting butterfly valves to 12-in. and larger HDPE pipe 10. DIP/IPS sizing causes some confusion

The workshops participants offered a number of strategies to overcome the joining method/fittings issues. These strategies are listed in Table 9.

Table 9. Strategies to address joining methods/fittings issues.

Strategy	Description
1	• Pipe manufacturers should consider offering fittings as well so that the utility is dealing with a single source for its needs
2	• Pipe manufacturers should consider providing fittings (either molded or fabricated) for larger pipe sizes • Solutions should be developed for connecting HDPE to valves and other pipe materials • Special orders should be minimized to the extent possible
3	• Pipe manufacturer should consider streamlining their product lines and reduce the variety of products offered (DIP/IPS size, various classifications) to reduce potential for confusion
4	• The experience gained in the gas experience should be shared with water industry
5	• Manufacturers and industry associations should consider offering training for design, installation, inspection, and maintenance of HDPE pipe
6	• Industry associations should consider providing certifications for utility and contractor personnel regarding handling, installation, joining and maintenance of HDPE pipe • Industry associations should also consider equipment certification
7	• There should be requirements developed by the industry for contractor qualifications and certification
8	• Development of training materials for trade school programs can improve the quality of installed pipelines
9	• Establishing a Center of Excellence can promote best Practices for HDPE pipe
10	• When connecting HDPE to another pipe, the end of HDPE pipe should be restrained by a thrust collar or otherwise restrained. If not, there is potential for the joint to pull open due to temperature effects.

CONCLUDING REMARKS

The Project Workshops provided valuable input to the project and assisted the Project Team to improve upon the project scope and experimental approach. The structured approach utilized for the workshops allowed the critical topics to be identified in an efficient manner. The limited and valuable time of participants was mostly devoted to discussion of the most critical topics. The workshops enabled the Project Team to explore different perspectives and identify several studies and experiences brought up by the Project Participants. Specifically, the following areas were explored in detail during the course of the three workshops organized by the project team:

- Perceptions issues related to use of HDPE for large diameter pipes and strategies to address those issues.
- Outstanding issues related to specifications, design, installation and maintenance of large diameter HDPE pipe and strategies to address those issues.
- Issues related to pipe joining and fittings and strategies to address those issues.

The following specific strategies were offered for the HDPE pipe industry:

- Establishing a Center of Excellence for HDPE Pipe to promote “Best Practices” for HDPE pipe.
- Documenting successful installations of HDPE pipe.
- Encouraging utilities that provide both water and gas service to use HDPE for water applications as they already may have an experience base with use of HDPE for gas applications.
- Encouraging contractors with gas pipe installation experience to serve the water market.
- Highlighting the advantages of HDPE pipe such as its leak free nature due to butt-fused joints.
- Sharing the experience of gas market with water market.
- Developing guidelines for design professionals, installers, inspectors, and operators of HDPE pipe.
- Developing “Best Practices” for all aspects of HDPE pipe.
- Developing guidelines for evaluation and condition assessment of HDPE pipe
- Developing and offering training to all professionals involved in the design, installation, inspection, and maintenance of HDPE pipe.
- Partnering with trade schools to train the required workforce.
- Developing and offering certification for various professionals involved in the design, installation, inspection, and maintenance of HDPE pipe.

The following specific recommendations were offered for utilities:

- Considering life cycle cost when selecting a pipe material.

- Utilizing Quality-based Selection (QBS) process to select qualified design consultants.
- Specifying an acceptable level of qualifications for contractors.
- Engaging qualified professionals to perform forensic evaluation of failure incidents to learn from the failure and ensure the root cause of failure is established and eliminated from future design.

The following specific recommendations were offered for pipe installers:

- Hiring trained personnel or offering full training and supervision for their personnel who may not be fully experienced.

The following specific recommendations were offered for pipe manufacturers:

- Streamlining of HDPE pipe product lines to reduce variety of products available and minimize confusion.
- Offering fittings as well so that the utility is dealing with a single source for its needs.
- Developing solutions for connecting HDPE to valves and other pipe materials.
- Offering regular field observations to promote best practices.

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It's a Blasting Good Time! Installation of a 30-inch HDPE Transmission Main in a Corrosive Environment, through Rock, under a River, and Adjacent to an Active Failing Pipe

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Abstract

Maintaining uninterrupted service and fire protection to customers is always a top priority for water providers. When a major transmission main begins to show signs of failure, addressing the cause and fixing the problem also become a top priority. When Howard County, Maryland began to experience multiple wire breaks in a major 30-inch Prestressed Concrete Cylinder Pipe (PCCP), they recognized the potential for a major service interruption to their customers and began to take steps to protect that asset. Howard County faced a unique challenge in that a new pipeline was needed that had to be resistant to stray current, installed through rock (directly adjacent to an existing failing PCCP main that was to remain in service during construction), and cross the Little Patuxent River. The result was the installation of 2,200 linear feet of a 3-inch thick 30-inch diameter HDPE water main that including blasting through rock and an open cut installation under the Little Patuxent River.

EXECUTIVE SUMMARY

Maintaining uninterrupted service and fire protection to customers is always a top priority for water providers. When a major transmission main begins to show signs of failure, addressing the cause and fixing the problem also becomes a top priority. When Howard County, Maryland began to experience multiple wire breaks in a major 30-inch Prestressed Concrete Cylinder Pipe (PCCP), they recognized the potential for a major service interruption to their customers and began to take steps to protect that asset.

An acoustical survey completed in 2003 revealed premature failures of the pre-stressed wires in a thirty year-old 30-inch diameter PCCP main that serves as a major transmission main feed for the County. It was determined that approximately 2,200 linear feet (LF) of main had the potential for failure if steps were not taken to protect the existing main or replace the pipe completely.

The first step in the process was to determine the reason for the wire breaks in the existing pipe. Studies revealed that stray currents from an existing underground cross-country natural gas pipeline with an impressed current system was severely damaging the main. If left alone, the corrosion caused by the stray current could potentially cause a catastrophic failure of the pipe.

Due to the potential increase in future demands, it was decided that a parallel main would be installed in addition to repairing the existing main. A complete materials analysis was performed in an effort to determine a suitable material for replacement given the existing stray current conditions. Multiple options were included in the analysis including traditional materials (Ductile Iron (DI), PCCP and Steel) as well as alternative materials (polyvinyl chloride (PVC) and high density polyethylene (HDPE)). Ultimately, it was determined that HDPE was the best option for this application. Following the selection of the material, an even more in depth review of HDPE pipe was completed, as it was a new material for the County, who historically used metallic or concrete pipe for the larger transmission mains.

During construction, the first challenge was logistics. Given an available 40-foot wide limit of disturbance (LOD) adjacent to US Route 29, in close proximity to a highly residential neighborhood and given 50-foot sections of three inch thick HDPE that had to be fused above ground, staging and sequencing were key factors. The Contractor opted to fuse multiple sections of pipe above ground, stage them onsite, then install in one long trench. Although a challenge from the start, this method proved to be quite successful.

The second challenge was that the new main was to be located parallel to and in close proximity to the existing, failing PCCP main that was to remain in service during the installation process. This posed a unique challenge in that a seismic refraction survey indicated the presence of rock that may require blasting. Due to concerns with regards to maintaining the structural integrity of the existing pipeline, a detailed monitoring plan was developed for use during blasting activities that included an allowable peak particle velocity. Details of this survey are included later in this paper. The contractor successfully blasted through rock for approximately one third of the length of the pipeline installation. Monitors placed on the existing pipeline indicated no damage during the installation.

Finally, installation of the new main required a major river crossing that was to be completed via open cut installation. The contractor opted for an alternative to the standard coffer dam in order to facilitate the crossing and used a PortaDam, which allowed for installation of the pipe across the full channel at one time. This technique was successful and the crossing was completed within one working day.

These unique challenges were all successfully overcome and the project was a blasting success!

OVERVIEW OF HOWARD COUNTY AND ITS WATER SYSTEM

Howard County is located in the central part of Maryland and borders six surrounding counties. Howard County purchases the majority of its potable water from the City of Baltimore which operates two (2) treatment plants (Ashburton and Montebello). The potable water is conveyed from the City of Baltimore through a series of transmission mains through Baltimore and Anne Arundel County's, and Howard County has three (3) large master meter connections into the County. The series of transmission mains which are the subject of this paper represent one of the main sources of supply into Howard County.

Howard County's water system consists of:

- More than 1,000 miles of water main
- Approximately 900 miles of Cast Iron / Ductile Iron Pipe
- Approximately 100 miles of PCCP/ Plastic Pipe
- Most transmission mains are PCCP and Ductile Iron
- Average Daily Requirement of 26 MGD
- 10MG of water storage

The backbone of their water distribution system is a series of transmission mains that parallel and cross under US Route 29, a highly traveled corridor that connects Baltimore to Washington, D.C. The transmission main that is the focus of this project is part of this backbone system as shown in Figure 1.

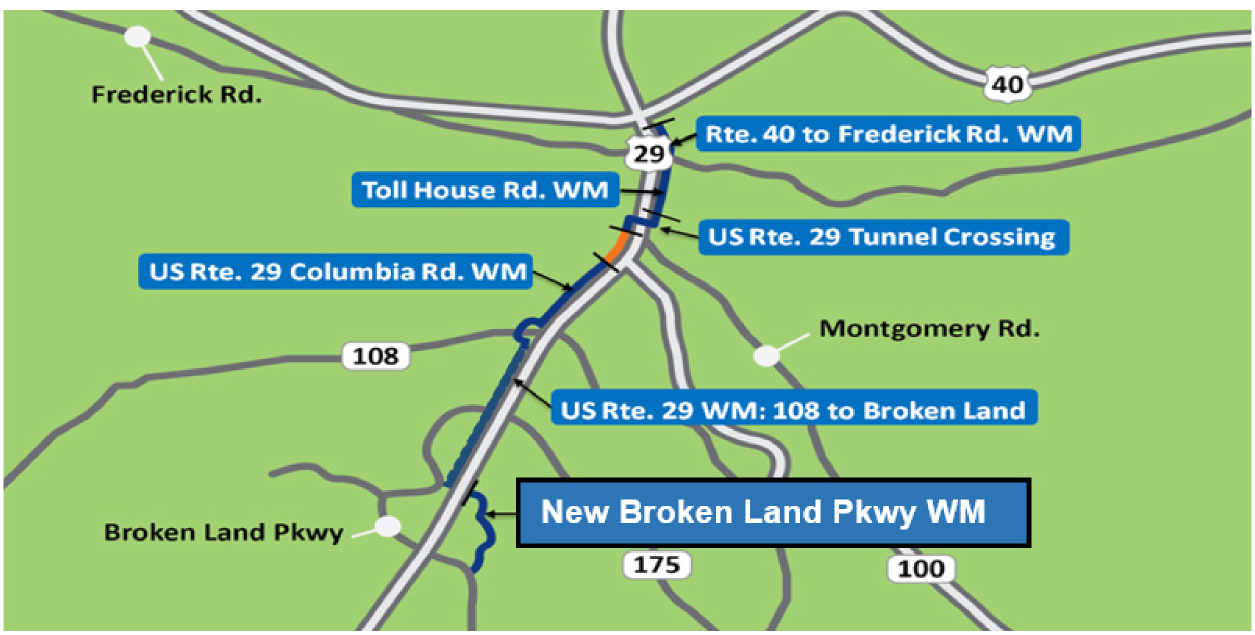

Figure 1: Howard County's Transmission Main Backbone

REPLACEMENT OF THE BROKEN LAND PARKWAY WATER MAIN

The Broken Land Parkway transmission main is one of the major water supply lines for the Owen Brown area in Howard County. The 30-Inch diameter PCCP water main was installed in 1975 and is a part of the County's critical backbone transmission main system that runs along US Route 29. The section of main included in the study of this paper runs cross country from River Meadows Road at US Route 29, paralleling and crossing the Little Patuxent River to just north of Owen Brown Road and Broken Land Parkway.

Initial Evaluation

In 2003, an acoustical survey revealed wire breaks in an approximately 1,000 LF section of the transmission main approximately 1,000 feet southeast of the intersection of River Meadows Road and US Route 29. Due to the critical nature of this transmission main, the County decided to evaluate the cause of the wire breaks further and ultimately conducted a corrosion study that revealed that two cross country gas mains (installed in 1941) were protected by an impressed current rectifier located along River Meadow Drive. It was determined that the localized damage to the pre-stressing wires on the existing PCCP main was likely being caused by the impressed current system. Although no pipe failures occurred, in 2005 the County pro-actively decided to replace the portion of the main that appeared to be at high risk for pre-mature failure.

An initial alignment evaluation was undertaken that included looking at replacement of the damaged section of pipe in place or parallel replacement. Replacing the damaged section in place would require the County to take the main out of service for an extended period of time. This option was ultimately eliminated due to the critical nature of this main. Other concurrent pipeline replacements in the County limited downtime for this main and it could not be taken out of service. Given that the County also was looking to increase redundancy and capacity in the area, it was recommended to install a 36-inch diameter parallel main in this area. This would allow the existing main to remain fully operational during the installation of the new section of pipe, while also increasing the available capacity.

Materials Analysis

As part of the initial evaluation, a materials analysis was also completed. The presence of the impressed current system on the existing gas mains meant that any pipeline installed with a metallic component could be at risk for corrosion and premature failure if not cathodically protected. As part of the materials analysis, both metallic and non-metallic materials were evaluated. The materials included ductile iron (DI), steel, PCCP, polyvinyl chloride (PVC) and HDPE. The existing main had an operating pressure of the main is 115 psi which was used as the basis for designing the new main. Table 1 shows the results of the headloss calculations completed for various pipe materials that were under consideration for the new main.

Table 1: Headloss calculations for various pipe materials

Pipe Type	Nominal Pipe Diameter (inches)	Inside Diameter (inches)	Coefficient of Friction	Length of Reach (ft)	Flow (fps)	Estimated Headloss (ft)	Flow (fps)	Estimated Headloss (ft)
HDPE DR-13.5	**42**	**37.512**	**130**	**3100**	**2.7**	**1.9**	**5.4**	**6.9**
HDPE DR-11	**36**	**30.91**	**130**	**3100**	**4.0**	**4.9**	**8.0**	**17.8**
Steel	**36**	**35.5**	**120**	**3100**	**3.0**	**2.9**	**6.1**	**10.5**
PCCP	**36**	**36**	**120**	**3100**	**2.9**	**2.7**	**5.9**	**9.8**
DIP	**36**	**37.24**	**120**	**3100**	**2.8**	**2.3**	**5.5**	**8.3**
HDPE DR11 versus DIP Comparison					2.6 feet = 1.1 psi		9.5 feet = 4.1 psi	

Manufacturers of these alternate pipe materials were contacted regarding the potential use of their products in this application. Each of the materials was available in the required size and could meet the design pressure. The County had great familiarity with both DIP and PCCP, having successfully installed them on previous transmission main projects. In addition, steel had also been bid competitively on previous County projects and the requirements were well known. The other materials had not been used as extensively on large diameter water main applications. The materials analysis included an evaluation of each material based on its applicability to this project, cost, and proven history for use on large diameter water mains.

DIP, PCCP and steel derive their ability to withstand internal pressure from their metallic construction and have historically been the go-to materials for large diameter mains. However, given the proximity of the impressed current system and the known corrosive issues associated with it, the use of metallic components on this project would have required some form of cathodic protection measures. In contrast, PVC and HDPE mains are non-metallic and are not subject to the damaging effects from stray currents. One concern with using PVC or HDPE pipe was the lack of history for use on large diameter potable water main projects in the mid-Atlantic region, including Howard County itself. Although each of the plastic pipe materials is subject to deterioration from chemical compounds - PVC degrades in the presence of benzene and HDPE is permeable for hydrocarbon based compounds, it was determined during field exploration of the area that there was no evidence of either benzene or hydrocarbons.

The cost evaluation for each of the alternative materials yielded a nominal difference in cost between the various alternatives, with DIP being the highest overall cost and PCCP the lowest.

However, given that the difference between the highest and lowest was 17%, this cost was not deemed to be the most critical factor in the recommendation. In fact, discounting DIP, the difference in cost was only 6%, with HDPE being the second highest cost.

Given the concern with potential corrosion issues, the plastic pipe options were deemed the best alternatives. Given that the pipe alignment was to parallel the existing main that mirrored the winding Little Patuxent issue, the selected pipeline material would be required to offer some flexibility as well. Given that PVC pipe is more limited in its ability to deflect at joints, when compared to HDPE, it would require many metallic fittings to meet the geometry required to parallel the river. Based on this, it was determined that HDPE was the best alternative and this material was further evaluated for use. Figure 2 shows the flexibility of the HDPE main, which contributed highly to it being considered the most viable material option for this water main.

At the initial time of the evaluation, 36-inch diameter (ID) HDPE pipe was not available in the required pressure class. However, based on the initial hydraulic evaluation, it was determined that in lieu of a 36-inch diameter main, a smaller 30-inch diameter (ID) main could be used. It was determined that due to the high coefficient of friction value of HDPE, the reduction in diameter was not an issue from a hydraulic standpoint.

Figure 2: Installation of the winding 30-inch HDPE water main

Geotechnical Evaluation

The existing main was known to be structurally unsound due to both age and corrosion. Given that the new main was to be installed within 20-feet of the existing main that is in very fragile condition, an extensive geotechnical investigation was completed to evaluate potential excavation techniques. As part of this investigation, a seismic refraction survey was completed that evaluated both the depth to rock as well as the competency of the rock. The survey included the transmission of sound waves into the subsurface and then recording the acoustic responses using a seismograph at set distances from a seismic enter source (i.e. hammering on an aluminum plate). The seismograph measured the time it took for the compressional sound wave generated by the energy source to travel down through the layers of the subsurface and back up to detectors (geophones) placed on the surface. By measuring the travel time of the sound wave, the subsurface geology was able to be interpreted.

As part of the survey, five transects along the pipeline were evaluated. The seismic data was collected using a 24-channel seismograph with twelve geophones. A sledgehammer hitting an aluminum plate was used as the energy source for this survey. Five hits were made for each geophone location and the results were recorded. The study indicated that friable rock with low rock quality data (RQD) values was present (consistent with the previously prepared geotechnical boring report). Based on the results of the above analysis, a maximum peak particle velocity was developed for use during blasting. The intent of this maximum peak particle velocity was to limit impact to the existing PCCP main. Provisions were included in the bidding documents to include monitoring of the existing main if the contractor opted to use blasting to install the new pipe. Figure 3 shows partial results of the seismic refraction study completed as part of this project.

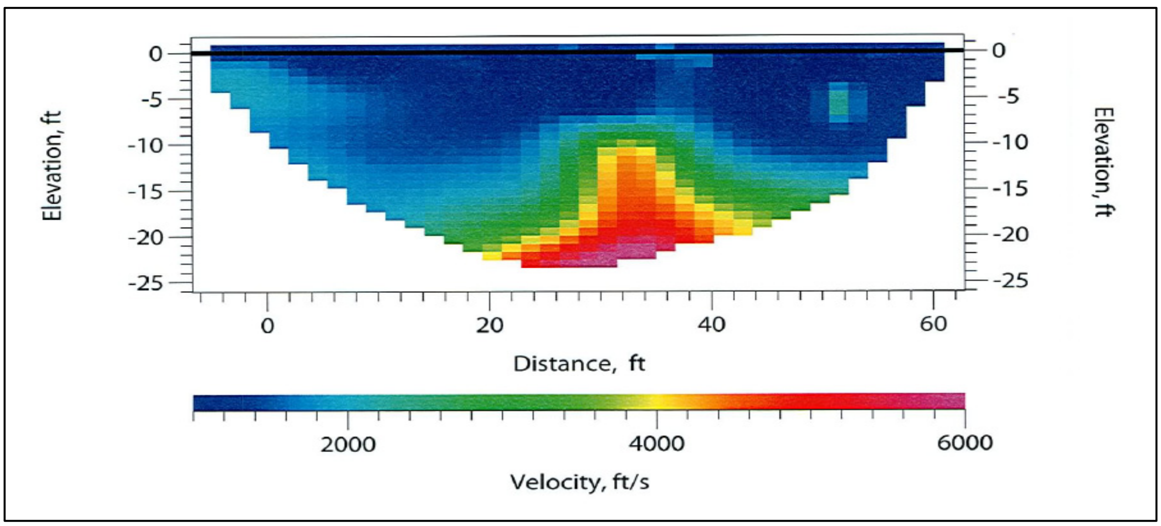

Figure 3: Partial results of seismic refraction survey completed along the proposed alignment

CONSTRUCTION CHALLENGES

Installing Forty Ton Sections of Pipe

Construction of large diameter pipelines pose unique challenges due to the special equipment required to maneuver and install the large materials. Normally, installation of pipe this large would mean digging a trench, using a trench box for support, lowering pipe into the trench, connecting the pipe segments and backfilling. However, given the nature of HDPE, this is not reasonable as the pipe segments have to be fused together using a fusion machine. The first challenge for installation of the Broken Land Parkway water main was determining how the pipe would be staged, fused, placed into the trench and backfilled. Complicating the installation is the relatively narrow limits of disturbance (LOD) for this project, which was 40-foot wide for staging, maneuvering, etc.

The 30-inch diameter HDPE used for this project has a wall thickness of 3-inches and was delivered in 50-foot lengths. Each length of pipe weighed in at approximately 10,000 pounds. The fusion machine required to join these pipe segments was quite large and it was quickly realized that lowering the machine into the trench to fuse each segment was not an option. As an alternative, the Contractor proposed fusing 400-foot segment of pipe (Figure 4), staging within the LOD, then rolling the pipe into the trench and backfilling, leaving a portion of the pipe above grade that would then be fused to the next 400-foot section. In order to maneuver the larger sections of pipe, a special steel chain-type strap was required, as the normal pipe straps could not support the weight of the fused sections (Figure 5).

Figure 4: Staging of 400-foot sections of pipe

Figure 5: Steel Strap used for lifting the pipe

Further complicating the installation of the new transmission main was a 100-year storm that swept through the area during the time of the pipe fusion process. Understanding that the construction site was adjacent to the Little Patuxent River and well within the 100-year floodplain, this caused widespread flooding of the site and damage to the equipment, including the fusion machine itself. Although the equipment was ultimately repaired and the site cleaned up, this unexpected disruption resulted in a significant delay in the schedule and costly repairs.

Blasting next to an active failing main

As discussed previously, the existing 30-inch PCCP main was in poor condition and was to remain in service during the construction of the new main. The contractor was directed to use extreme caution when working in the vicinity of the fragile transmission main. As indicated previously, during the design of this project, an extensive geotechnical investigation identified the presence of friable rock and a report was developed that included provisions for the peak particle velocity that could be used without compromising the integrity of the existing PCCP main.

Prior to construction it was determined that blasting would be required in order to install a portion of the pipeline that ran cross country through the woods. A rock profile of the area showed isolated pockets of hard rock that could not be removed using standard equipment. A controlled blasting plan that included short lengths of blasting using low-voltage charges was developed by the Contractor. The charges were placed in the trench along the path of the new pipe, then detonated and the rock removed from the site. The area around the existing pipe was monitored by the contractor for movement.

The contractor provided seismic monitoring of the ground surrounding the existing pipe to confirm movement near the failing main. In addition, the County also closely monitored the condition of the existing main as part of their acoustic fiber optic (AFO) monitoring program, as the existing main was already being monitored for pre-stressed wire breaks. If ground movement was recorded near the existing main, the Contractor was to halt the blasting and re-evaluate the plan. In addition, if damage to the existing pipe was recorded as part of the AFO monitoring program, the provisions of the construction documents required that it be repaired at the expense of the contractor, who was on-call 24-hours a day during the blasting operations.

The contractor successfully executed the removal of the rock without damaging the existing pipe. The upfront identification of the potential rock by the County during design and the inclusion of the blasting and monitoring requirements in the construction documents saved them what could have been a costly change order at construction.

Crossing the River

Working in an environmentally sensitive area poses a challenge – install the pipe successfully, while minimizing impacts to the surrounding environment. The new transmission main parallels, and then crosses the Little Patuxent River. During the design process, coordination with the Maryland Department of the Environment (MDE) allowed for an open cut crossing of the river, in lieu of trenchless installation. At the point of the crossing, the river channel measured just over 50-foot wide from bank to bank. This meant a significant cost savings to the County, as trenchless installation can often become a costly method due to the specialized equipment, training and personnel required.

The crossing was originally to include a standard stream diversion; however, due to the nature of the 50-foot HDPE pipe segments, the crossing required multiple segments in order to fully cross the river. This meant that a standard stream diversion, that would typically divert flow from one side of the river at a time, could not be used, as it required half of the river be bypassed at a time in order to facilitate installation. Since the pipe could not be fused in the middle of the river, the contractor opted to use an alternate means of diversion for the river – a PortaDam with a flume bypass (Figure 6). This method allowed for the full width of the river to be diverted through two 36-inch pipes, leaving the channel dry for a 50-foot long section and allowing for the open cut installation of the pipe. The channel was then restored to its natural condition and the bank stabilized using the same rock previously removed from the river bottom.

Figure 6: Open cut river crossing using PortaDam and flume bypass

CONCLUSION

With this main now successfully replaced, the County plans to take the existing 2,200 feet of PCCP main out of service to complete a full pipeline condition assessment on the damaged section to determine if the pipe can be rehabilitated using trenchless technologies.

Howard County prides itself on being pro-active in their approach to managing their water distribution and transmission system. Their pipeline condition assessment program allows them to monitor their large diameter transmission main pipes – the backbone of their distribution system. This means they minimize the likelihood of water main breaks that could cause interruption in service to a large portion of the County's residents and thereby improve the quality of their service overall.

The County also takes a progressive approach to the use of alternate pipe materials and installation techniques, allowing them to be knowledgeable in the latest technologies and cost-saving techniques when it comes to water distribution, making them leaders in the local Baltimore area.

Evaluation of Corrugated HDPE Pipes Manufactured with Recycled Content underneath Railroads

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Abstract

Corrugated high-density polyethylene (HDPE) pipes are an attractive product for culvert and drainage applications in the railroad industry due to their resistance to corrosion and abrasion, long service life, and flexibility. Railroad specifications currently require these pipes to be manufactured with 100% virgin materials. However, due to the push for more sustainable and cost-effective engineering materials and practices, the railroad industry would benefit from using pipes made with recycled content provided their long-term performance was equivalent to pipes made with virgin materials. To evaluate the performance of corrugated HDPE pipes made with recycled content in rail applications, a pilot study was conducted with the Southeastern Pennsylvania Transit Authority (SEPTA). The study was funded cooperatively by SEPTA and the National Cooperative Highway Research Program (NCHRP) Project 4-39. Two 30-inch diameter corrugated HDPE pipes, one manufactured with 100% virgin materials and one manufactured with post-consumer recycled content, were installed underneath a regional commuter rail line in northeast Philadelphia with 2 feet of cover from the top of the pipe to the bottom of the railroad tie. The pipes were instrumented with strain gages and extensometers to record live-load data and monitor the pipes over time. A laboratory study was also developed to assess the long-term durability of pipes made with recycled content with regards to cyclical live loads. The pipes have been in service for over 1 year and are performing as designed, with no change in performance since the date of installation. This was a groundbreaking study as it included the first corrugated HDPE pipe manufactured with post-consumer recycled content installed underneath one of SEPTA's regional commuter lines. The research project is a key component of SEPTA's ongoing sustainability initiatives.

BACKGROUND INFORMATION

Corrugated HDPE pipes have been used for decades in culvert and storm drain applications and are considered an attractive product for railroad applications due to their durability and corrosion resistance. Most of the pipes currently used in these applications are manufactured with 100% virgin materials. Recently the highway and railroad industries have expressed an interest in utilizing pipes made with recycled content due to their environmental and economic benefits. To study the performance of pipes made with recycled content, SEPTA and NCHRP cooperatively funded this research project.

Corrugated HDPE pipes manufactured with recycled content have been successfully used in the agricultural and highway industries for various drainage applications. However, the cyclical loads in railroad applications are greater than those in typical agriculture and highway applications, so we wished to evaluate the long-term effects of these cyclical loads on the performance of the pipe. Historically, the corrugated HDPE pipe industry has observed no fatigue-related failures in railroad applications for pipes made with 100% virgin materials. However, pipes manufactured with post-consumer recycled (PCR) materials can have a greater likelihood of containing contaminants (e.g. label remnants, other non-PE plastic materials, etc.) than pipes made with virgin materials. The majority of these contaminants are filtered during the washing and pelletizing process at the recycling facility, and furthermore by screen changers in the extrusion process at the pipe manufacturing plant. While stress-crack failures related to contamination have not been observed in pipes manufactured with recycled content for agricultural or highway applications, we desired to study their performance under the more severe loading conditions in railroad applications.

The research consisted of two primary components. First, a field study was conducted to determine the magnitude of loads in these applications. Second, an accelerated laboratory test was conducted to evaluate the long-term service life of the materials relative to fatigue loading.

FIELD STUDY

The Villanova University research team worked collaboratively with the Southeastern Pennsylvania Transit Authority (SEPTA) to identify a location for a test installation for pipes made with recycled content underneath a live commuter railroad. The location identified was in New Britain, PA, approximately 30 miles Northwest of Philadelphia.

80 feet of pipe were donated by Advanced Drainage Systems for the project. 40 feet (2 sticks of pipe) were made in accordance to AASHTO M294 and contain no recycled content, and another 40 feet (2 sticks of pipe) were made in accordance to ASTM F2648 with 49% PCR content, 49% virgin HDPE materials, and 2% carbon black (carbon black is required for UV protection). Both pipes were evaluated

according to their respective material standards, and the results are summarized in Table 1.

The bell end of the AASHTO M294 virgin pipe was instrumented with strain gages and extensometers to measure strain and deflection, and the spigot end of the ASTM F2648 pipe was similarly instrumented (Figure 1). The pipes were installed with the joint under the mid-point of the track and the instrumentation directly under each rail, as shown in Figure 2. A watertight gasket was used at the joint, though watertight performance was not a requirement by SEPTA for this application.

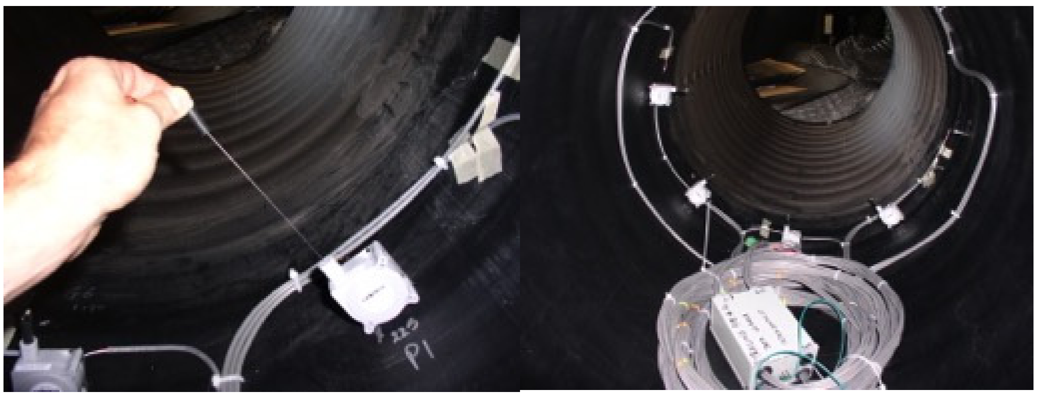

Figure 1: Instrumentation of test pipes

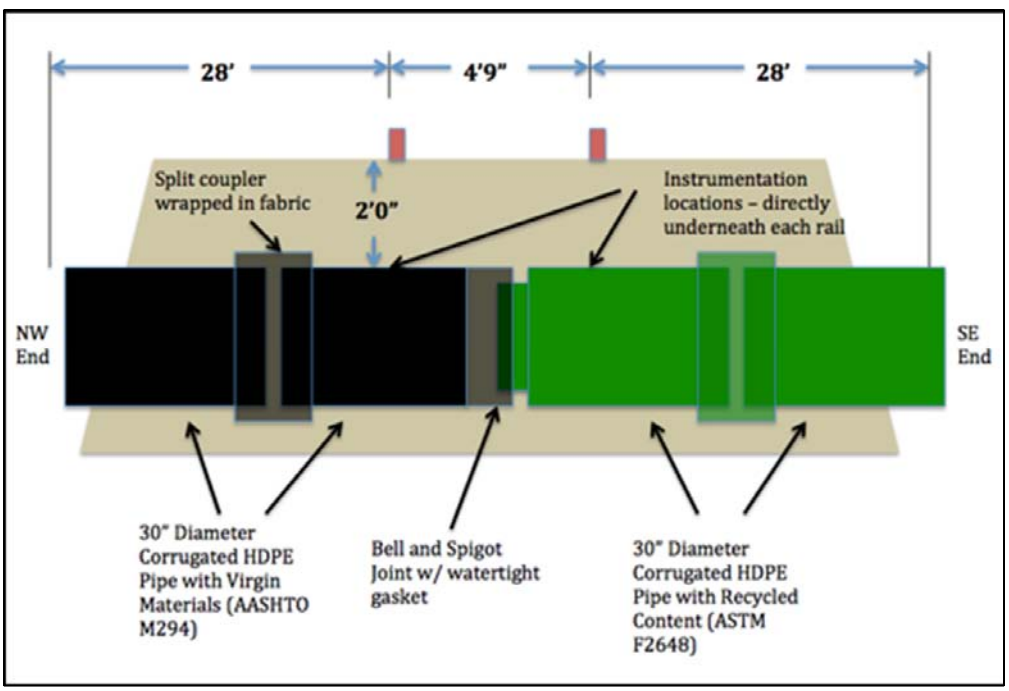

Figure 2: Schematic of field test installation

Table 1: Properties of test pipes

Property	Test Method	AASHTO M294 Pipe (100% Virgin)	ASTM F2648 Pipe (49% PCR)
Pipe plaque density	ASTM D 1505	0.963 g/cm^3	0.966 g/cm^3
Melt index	ASTM D 1238	0.12 g	0.30 g
Carbon Black %	ASTM D 1603	2.15 %	2.57 %
Flexural Modulus	ASTM D 790	152,755 psi	146,322 psi
Yield Strength	ASTM D 638	4,050 psi	4,062 psi
Pipe liner NCLS	ASTM F 2136	87.9 hrs	18.4 hrs
Pipe plaque NCLS	ASTM F 2136	106.1 hrs	13.7 hrs
Pipe Stiffness	ASTM D 2412	35.0 lb/in/in	34.3 lb/in/in
Pipe Flattening	ASTM D 2412	> 20%	> 20%
Brittleness Test	ASTM D 2444	Pass	Pass

The pipes were installed in accordance to SEPTA installation practices. The existing track was removed and the trench excavated for the installation of the new pipes, as shown in Figure 3. Sloped trench walls were used in accordance to SEPTA practices. The trench width at the bottom of the trench was 8 feet, which allowed 2.5 feet on each side of the installed pipe to ensure adequate compaction of the backfill materials.

Figure 3: Removal of track for pipe installation

SEPTA's design requirements specified a minimum of 5.5 feet of cover from the top of the pipe to the bottom of the railroad tie. However, SEPTA engineers allowed this minimum to be reduced to 2 feet for the purposes of this test project. This was advantageous for the project as it allowed us to observe the behavior of the pipe in a more extreme installation.

Approximately 1 foot of bedding material was installed in the bottom of the trench. This material was classified as a modified 2-A granular material according to PennDOT specifications (mix of course stone, no greater than 2 in., and fines), and was the same material that was placed in pipe envelope. The backfill was compacted with a Wacker vibratory plate soil compactor in 8 – 12-inch lifts up to the springline of the pipe, then compacted in 12 – 24-inch lifts until the backfill extended to 1 foot above the pipe. A vibratory rammer was used to compact the backfill materials in the haunches of the pipe, and a Wacker vibratory sheepsfoot trench roller was used to compact the lifts after the backfill material reached the springline. The track ballast material (standard SEPTA ballast) was then placed on top of the modified 2-A granular material. The construction and pipe backfill process are detailed in Figures 4 and 5. We did not verify the compaction level of the backfill materials, but followed standard SEPTA installation practices.

Figure 4: Preparation of trench and lowering of pipe

Figure 5: Installation of pipe and railroad track

The center joint that joined the virgin M294 pipe with the ASTM F2648 pipe was a bell and spigot style joint. This joint was located at the center of the two track rails, as shown in Figure 2. Additional ASTM F2648 pipe was added to the southeast end of the pipe to daylight the pipe to the end of the trench. Similarly, additional M294 pipe was added to the northwest end of the pipe. These end pipes were coupled with split couplers and the joints were wrapped in fabric to evaluate an alternative joining method than the standard bell and spigot joint.

Pipe wall strains and deflections were recorded prior to and after construction for both the virgin M294 pipe and the ASTM F2648 pipe made with recycled materials. We noticed some slight peaking of the pipe due to the compaction of the backfill lifts. This is not unusual for quality installations. The deflection measurements are shown in Table 2.

Wall strains and deflections have been recorded on a quarterly basis since the installation of the pipes in October of 2013. Table 2 shows a summary of the data. Approximately 36 trains pass over the pipe each day, and each train has 3 – 6 cars. The SEPTA railcars are 85 feet long and weigh 150,000 lbs unloaded. Each railcar has 4 axles (two trucks, each with two axles) with a capacity of 109 passengers. See Figure 6 for an illustration of the SEPTA railcars. Figures 7 and 8 show typical strain and deflection readings in the pipe during a pass from a 3-car train. You will note 6 major peaks – the first occurring when the leading truck of the first car passes over the pipe, followed by the trailing truck of the first car and the leading truck of the second car, then the trailing truck of the second car and the two trucks on the third car. Also, note that each peak consists of two small peaks, one from each of the two axles on each truck. The sampling frequency of the data acquisition system is set to 50 Hz to ensure we catch all of the data from the cars, which are traveling at approximately 50 mph.

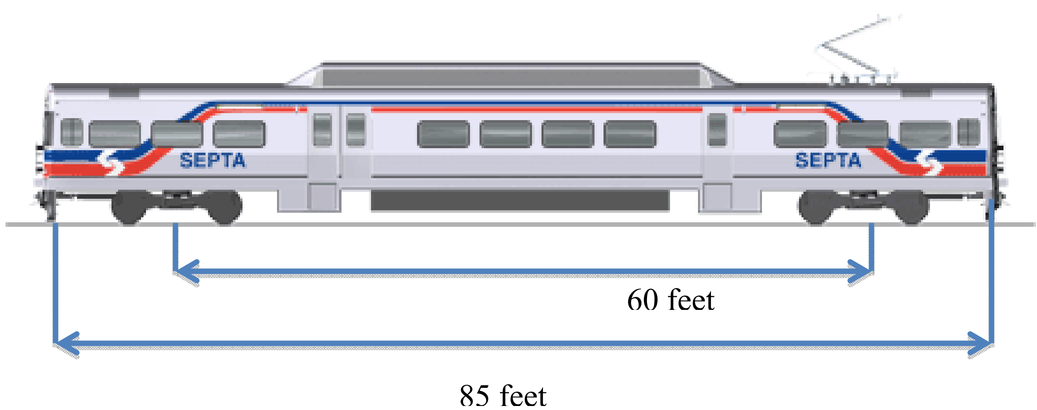

Figure 6: Typical SEPTA passenger railcar

Table 2: Measurements of field test pipes

Property	AASHTO M294 Pipe (100% Virgin)		ASTM F2648 Pipe (Recycled Content)	
Initial ID (in.)	30.1		30.1	
Installed Deflection (in)	V: 30.3	H: 29.9	V: 30.3	H: 29.9
6-month Deflection (in)	V: 29.9	H: 29.8	V: 30.2	H: 29.8
1-yr Deflection (in)	V: 29.9	H: 29.8	V: 30.3	H: 29.8
Max. Peak-Peak Dynamic Defl. (in)	< 0.0200		< 0.0200	
Max. Peak-Peak Dynamic Strain	500 μstrain		500 μstrain	

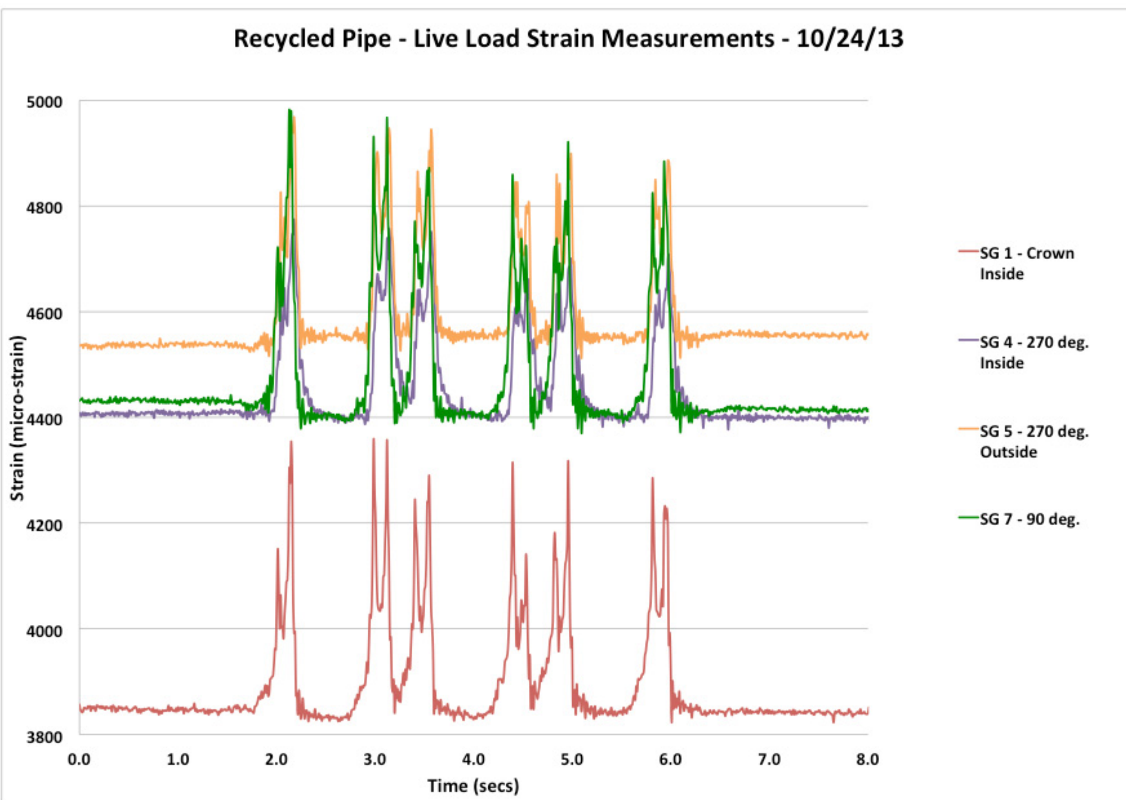

Figure 7: Typical live load strain measurements in recycled pipe

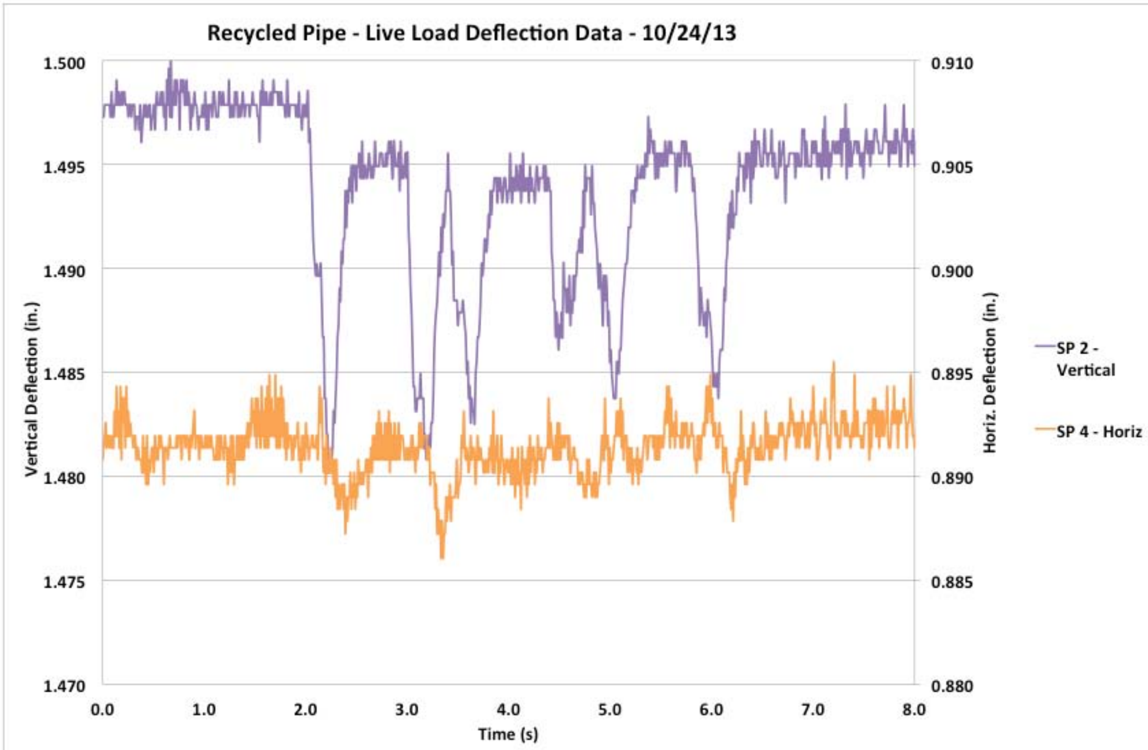

Figure 8: Typical live load deflection measurements on recycled pipe

LABORATORY STUDY

Test specimens were cut directly from the end sections of the pipes and instrumented with strain gages. The strain gages were installed at the junction where the liner meets the outer corrugation, as this is deemed a worst-case location for potential fatigue related failures due to the stress riser created by this junction. See Figure 9 for an illustration of the strain gage locations. Both the inner and outer walls of the test specimens were strain gaged. The instrumented test specimens were positioned in an MTS Universal Test Machine in Villanova University's Structural Engineering and Research Laboratory. Figure 10 shows the test specimens positioned in the universal test machine.

The test specimens were cycled at 5 Hz (0.2 second per load cycle) on the MTS test machine. Note that for a typical SEPTA railcar traveling at 50 mph (73 ft/sec), the average dynamic loading rate over the pipe is approximately 1.5 Hz (6 peaks observed in 4 seconds) for a given train, though the time between two adjacent axles on a truck is approximately 0.1 second. The laboratory test was accelerated in that the specimens were continually cycled, while the field pipes only see dynamic loading for 8 – 12 seconds every hour.

A calibration test was done to determine how the grip displacement related to the measured strain on the specimen, and a factor of 2 was applied to the dynamic strains observed in the field to simulate a worst-case condition. A grip displacement of 0.002 in. corresponded to a strain reading of 600 microstrain in the test specimen, so the peak-peak strain measurements in the lab were 1200 microstrain based on a grip displacement of +/- 0.002 in. The specimens were preloaded to 1500 microstrain prior to cycling. See Figure 11 for a typical trace of the strain measurements.

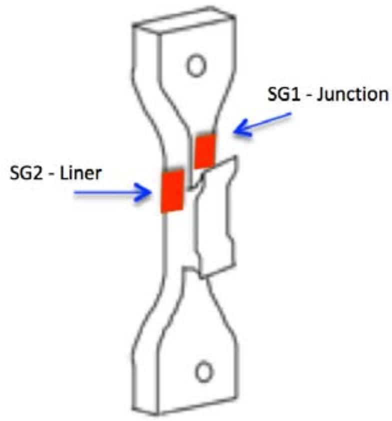

Figure 9: Illustration of strain gage locations on test specimen cut from pipe wall

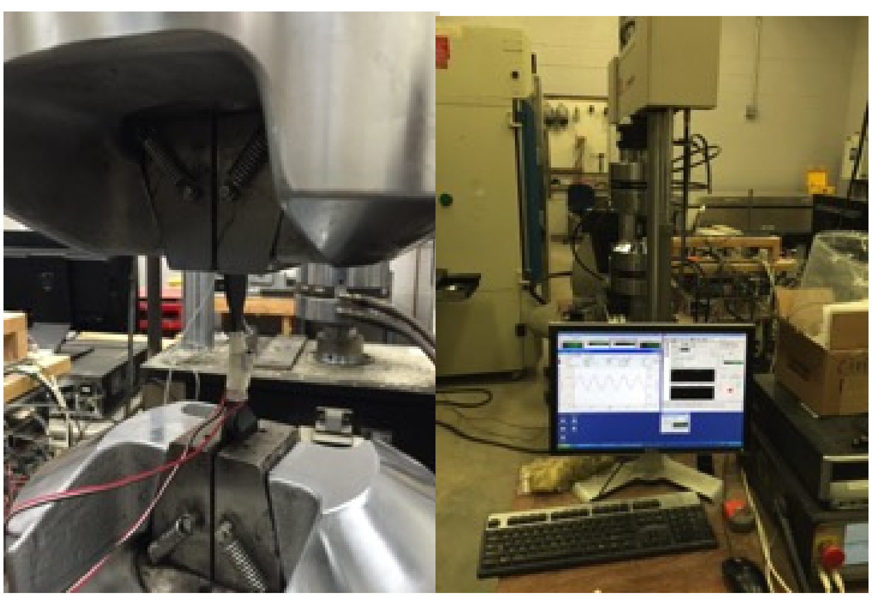

Figure 10: Instrumented test specimen in fatigue test

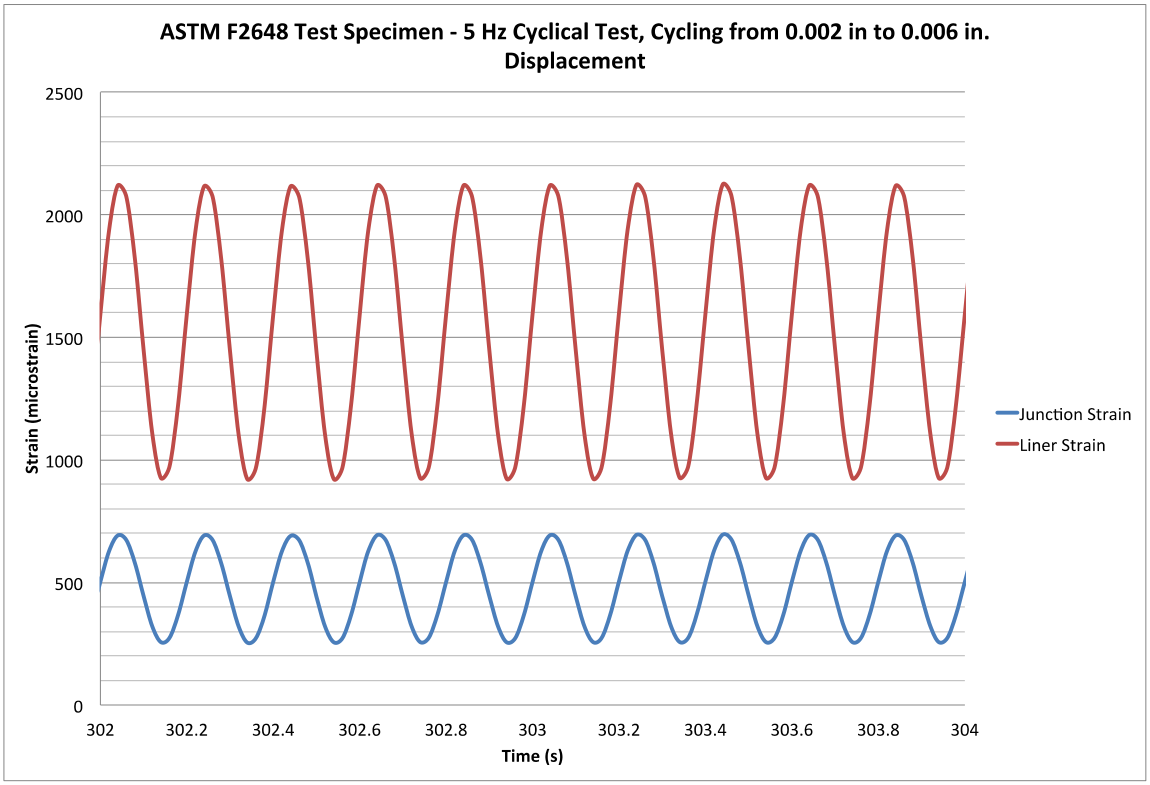

Figure 11: Strain measurements during cyclical fatigue test

There was some concern regarding potential heat generation in the test specimens due to the continuous cycling frequency. The research team conducted a test to measure the temperature of the test specimens at various frequencies and amplitudes. Figure 12 shows the results of this study. Based on this, we determined that 5 Hz is an appropriate frequency for testing considering the very low displacements (+/- 0.002 in.) in our test specimens.

Testing is still ongoing, but to date we have achieved over 2 million cycles on the test specimens manufactured with recycled content with no stress cracking observed. This is equivalent to over 20 years of service on typical regional commuter rail lines.

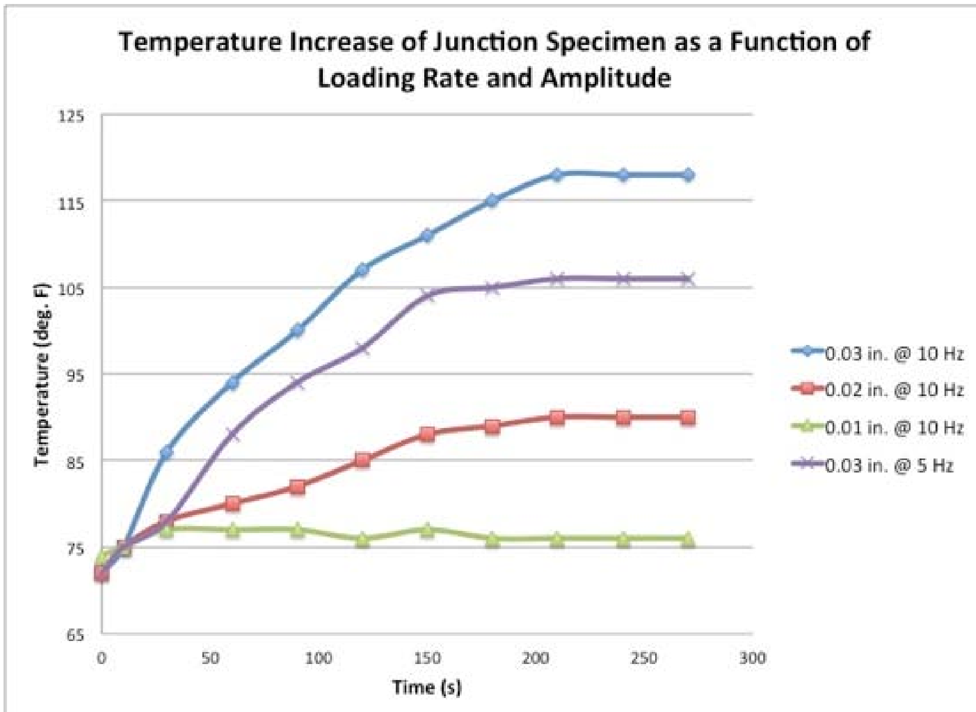

Figure 12: Temperature effects of various cycling rates on HDPE specimens

CONCLUSIONS

Corrugated HDPE pipes manufactured with recycled content can offer economic and environmental benefits for the highway and railroad industries. SEPTA and NCHRP collaboratively funded a study to evaluate the performance of these pipes relative to fatigue loading. Two test pipes were installed underneath a SEPTA regional commuter rail line in New Britain, PA. One of the pipes was manufactured with 100% virgin material, and the other pipe contained 49% PCR content. After 1 year of service, both pipes are performing well with no discernible differences noted. The measured strains and deflections on both pipes are minimal and well below industry recommendations. Additionally, a laboratory durability test to evaluate the pipes with regards to cyclical loads has indicated that the fatigue does not appear to be a concern for these pipes, even at very shallow fill heights.

Survey of Water Utilities on Their Experiences with Use of Large-Diameter HDPE Pipe for Water Main Applications

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Abstract

Large diameter water transmission pipelines are critical elements of water supply systems, because a failure can be catastrophic. Extended service interruptions for many customers, along with high costs of emergency repairs, inconveniences to general public, and associated water quality concerns can be results of large diameter pipeline failures. This paper presents a survey of water utilities for using large diameter (16 in. and larger diameter) high density polyethylene (HDPE) pipes for water transmission applications as a part of a wider-scale research project on durability and reliability of HDPE pipes (Water Research Foundation (WaterRF) #4485). While a full evaluation of a particular pipe material requires many parameters to consider, the main goal of this paper is to present the overall experience of water utilities with large diameter HDPE pipes. The survey of water utilities indicated that majority of respondents were satisfied with the durability and reliability of large diameter HDPE pipe, while five percent were unsatisfied. Survey respondents expressed concerns about tapping, repairs and joints. They considered permeation and oxidation to be minor concerns with no failures reported due to oxidation or permeation in large diameter HDPE piping systems. They also mentioned that some measures are required to improve construction techniques.

INTRODUCTION

The Report Card on America's Drinking Water Infrastructure states that U.S. infrastructure is in poor condition (ASCE 2013). Approximately 33 percent of drinking water is lost each year (Radoszewski, 2009). Due to leaks and breaks, water utilities in the United States lose more than 30 billion dollars' worth of drinking water between treatment plants and taps, and approximately six billion dollars per year needed to stop this loss (Jeyapalan, 2007). The water pipe rehabilitation costs may reach more than $1 trillion in the coming decade (ASCE, 2013). The large diameter (16 in. and larger) water pipe market in the U.S. mainly includes steel pipe (SP), prestressed concrete cylinder pipe (PCCP), ductile iron pipe (DIP) and PVC pipe. A study conducted by the Center for Underground Research and Education (CUIRE, 2013), 21 U.S. water utilities, serving a population of approximately 14 million, reported a small inventory of large diameter HDPE pipe, compared with other pipe materials. Pipe sizes for all materials ranged from 24 in. to 54 in. The large diameter HDPE pipe had an age of less than 25 years.

Recent advancements in polymer science has resulted in the production of high performance large diameter (16-in. and larger) high density polyethylene (HDPE) pipes capable of withstanding high pressures (Sever et al, 2014). However, the use of HDPE pipe for large diameter water applications has been limited due to inadequate experience and perception issues (Najafi, et al, 2015). As such, this paper presents a survey of water utilities for HDPE pipes (16-in. and larger) for water transmission applications as a part of a wider-scale research project on durability and reliability of large diameter HDPE pipe (Water Research Foundation #4485).

The survey questions included population served by utilities, HDPE type (3408, 3608 or 4710), age, and diameters currently in service, leakage issues, installation methods, life cycle costs, causes and modes of ruptures, restriction in usage, and overall experiences with durability and reliability of large diameter HDPE. The survey results stated that majority of water utilities were satisfied with use of HDPE pipe; however, some utilities had concerns regarding its maintenance and connections. This paper presents the details and results of survey, which will be of interest to pipeline professionals to understand water utilities' experiences with HDPE transmission mains.

METHODOLOGY

The survey questionnaire was prepared by the Center for Underground Infrastructure Research and Education (CUIRE), and was submitted to water utilities, as a separate part of WaterRF research project #4485, using a commercial survey Website. Out of 300 submitted surveys, 96 responses were received, but 39 only respondents stated they have large diameter HDPE. Out of those 39 respondents who had large diameter HDPE, most of them fully completed the survey, and some respondents partially completed the survey, as indicated with different number of responses in the following sections.

SURVEY RESULTS

Large Diameter HDPE Footage

Only 41% (out of 96 respondents) or 39 of water utilities had large diameter HDPE, and the remaining 59% either used smaller HDPE diameters, or never experienced with HDPE pipes. However, survey analysis is based on different number of responses actually received for each question (respondents did not answer all the questions).

Population Served

Figure 1 presents overall distribution of population served among survey respondents in each state. The highest number of population served by water utilities with large diameter HDPE pipes was in Texas with 4.6 million people. The second highest population was Colorado followed by California and Maryland. The lowest population served was in Oregon with Arkansas and Louisiana with slightly higher populations.

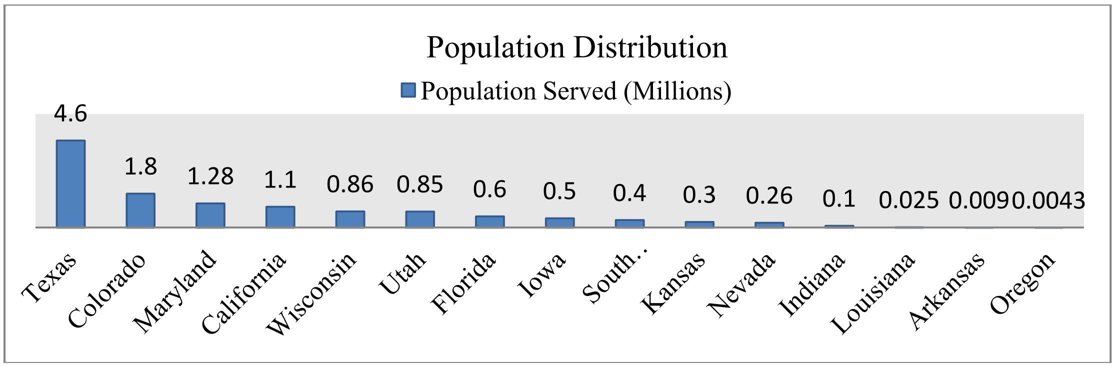

Figure 1. Population served by responding water utilities (based on 29 respondents).

HDPE Pipe Age Distribution

The majority of reported large diameter HDPE pipe in operation was made from resins classified as PE4710, and has been installed recently (within 5-10 years ago). Figure 2 illustrates the age distribution of HDPE pipes. It should be noted that some survey respondents were not confident about type of HDPE (PE4710 or PE3608/3408) in their system, however, it can be concluded that most recent large diameter HDPE pipe installations are PE4710. The confusion in PE4710 or PE3608/3408 may have impacted other survey responses as well.

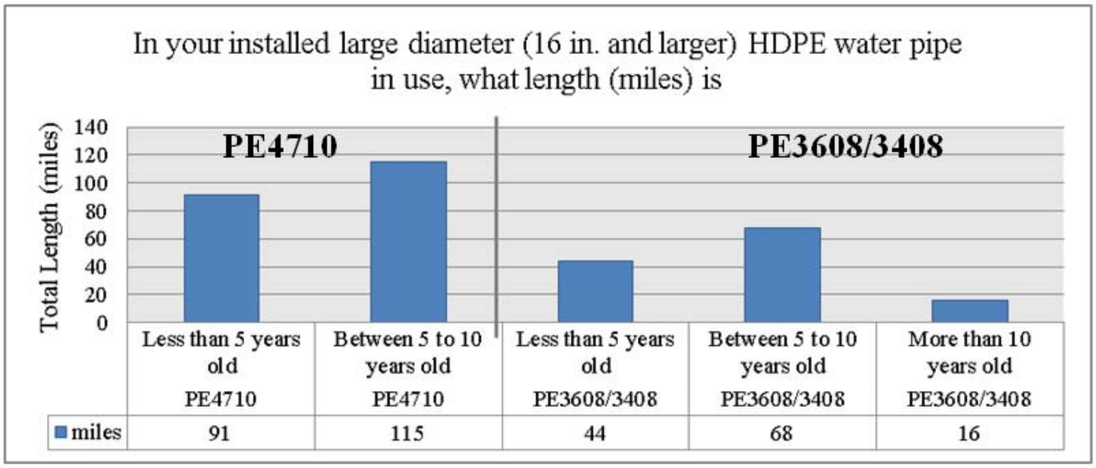

Figure 2. Number of respondents reported age distribution of HDPE pipes (based on 31 respondents).

Pipe Diameter Distribution

Most of the respondent water utilities have used PE4710 and PE3608/3408, in 16 in. to 24 in. diameters compared to diameter larger than 24 in. Figure 3 illustrates HDPE pipe diameter distribution.

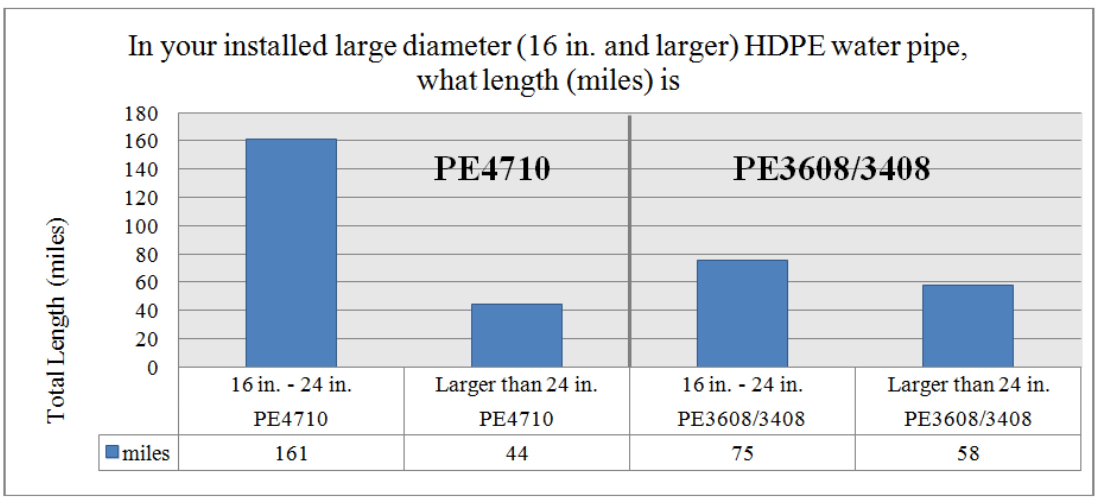

Figure 3. Number of respondents reported diameter distribution of HDPE pipes (based on 30 respondents).

Types of Permitted HDPE Pipes

Figure 4 present types of HDPE pipe diameters permitted in the responding water utility districts. Most of the responding water utilities used PE4710 compared to PE3608/PE3408. It should be noted that some water utility responded differently for specific diameters, so there are multiple diameter responses for each water utility.

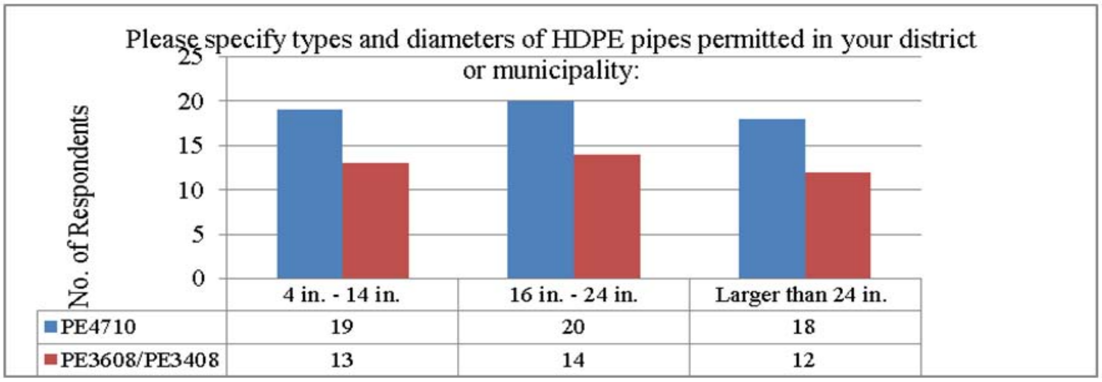

Figure 4. Number of respondents reported types and diameters of permitted HDPE pipes (based on 32 respondents).

Leakage

Approximately one third (9 out of 31 water utilities) reported having seen a leak at least in one of their HDPE water main systems. Some respondents indicated that leaks were primarily caused by improper construction methods with majority of leaks from third party damage. Other water leak causes, as stated by water utilities, were:

- HDPE fittings, joints and flanged adapters to DIP joints.
- Damage from other contractor's equipment.
- Flooding and washing out a river crossing.
- Faulty service saddles.
- Failure at manholes and service connections.
- Improper welding of joints.
- Pipe punctures during construction.
- Third-party damage.

Causes/Modes of Rupture/Leakage for PE4710

Among several causes, the survey results indicated that third party damage, installation defects, joint rupture, and fittings, are the major parameters that need to be considered for 16 in. to 24 in. for PE4710 pipe. On the other hand, for pipe sizes larger than 24 in., installation defect was a major issue and the main concern. Majority of water utilities reported no leaks in their system with following comments included in their responses:

- No problem with all these factors.
- No leakage in HDPE 16 in. and larger pipe.
- Our large diameter HDPE pipe has been installed less than 5 years ago, and we have had no failures.
- No pipe failures.
- Pipe has been installed less than a year and no rupture/damage was observed.

Causes/Modes of Rupture/Leakage for PE3608/3408

The survey analysis indicated that major issues were third party damage, installation defects, manufacturing defects, and fittings for 16 in. to 24 in. pipes. For 24 in. and larger, installation defects, fusion, electro-fusion, fittings, and third party damage were the major issues.

Concerns and Issues for Using HDPE Pipe

Figure 5 illustrates that highest critical concerns for PE4710 were repairs, tapping, and ease of use. Figure 6 illustrates that critical concerns for PE3608/PE3408 were tapping, repairs, joints, and ease of use. For both PE4710 and PE3608/PE3408, cracking, permeation and oxidation are rated the least issue and concern.

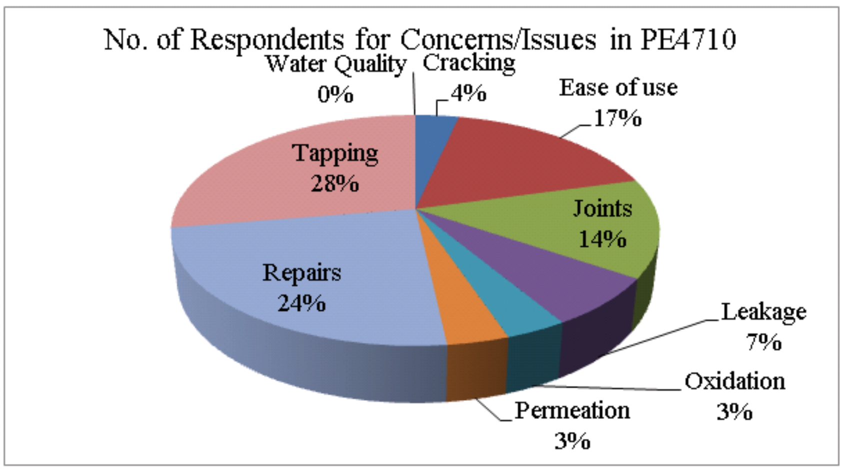

Figure 5. Concern/issues for PE4710 (based on 22 respondents).

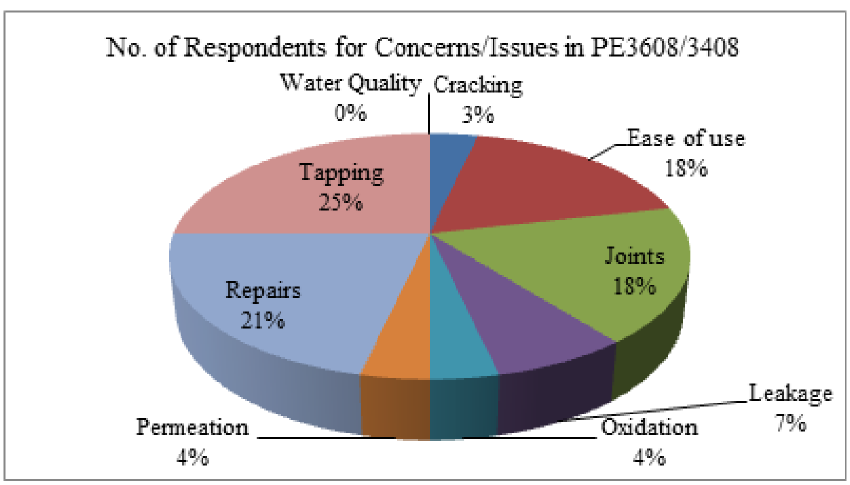

Figure 6. Concerns/issues for PE3608/3408 (based on 22 respondents).

Life Cycle Costs

Figure 7 illustrates that the most important factors impacting life cycle cost of HDPE pipes were "ease of maintenance," and "maintenance costs," followed by "life expectancy," "leak free joints," and "ease of tapping." Similarly, Figure 8 illustrates that "ease of maintenance," "ease of mechanical joints," and "ease of tapping," were most important factors for PE3608/PE3408.

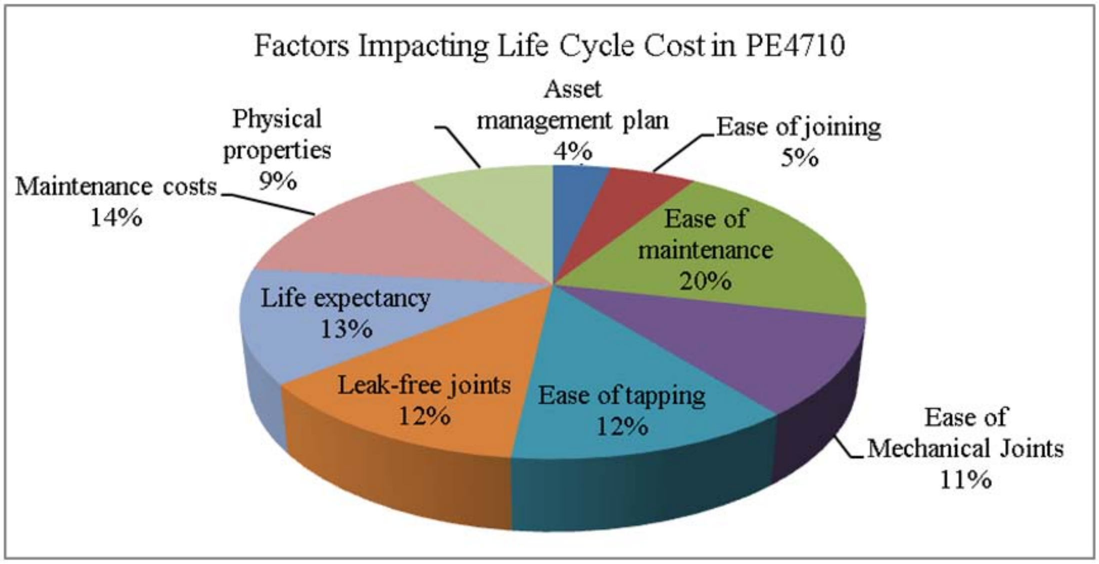

Figure 7. Factors impacting life cycle costs for PE4710 (based on 26 respondents).

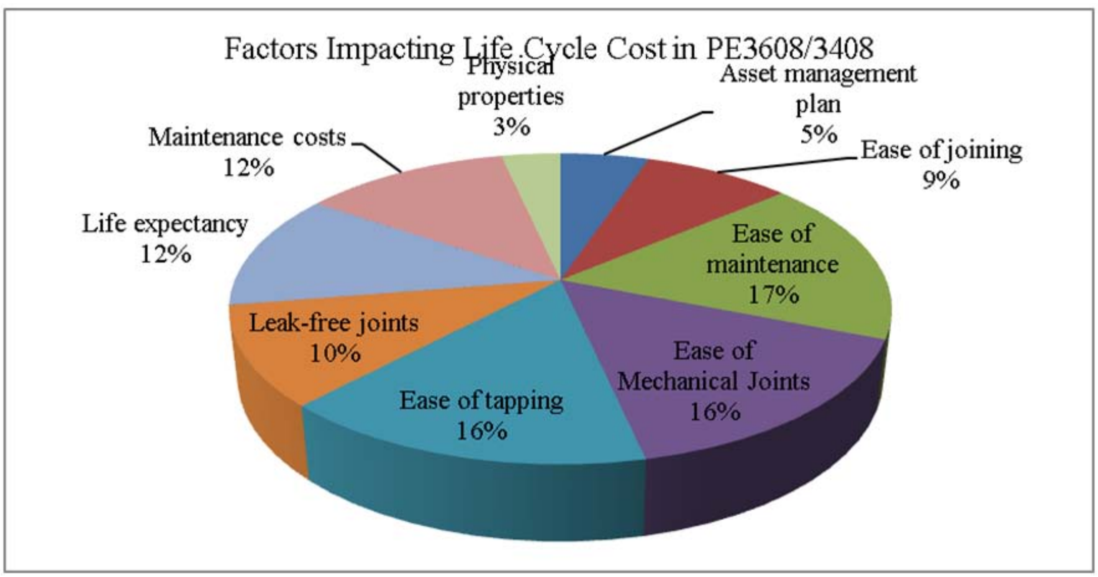

Figure 8. Factors impacting life cycle costs for PE3608/3408 (based on 26 respondents).

Rating Durability and Reliability of HDPE Pipe

According to responding utilities, and as shown in Figures 9 and 10, PE4710 is more durable and reliable than PE3608/3408.

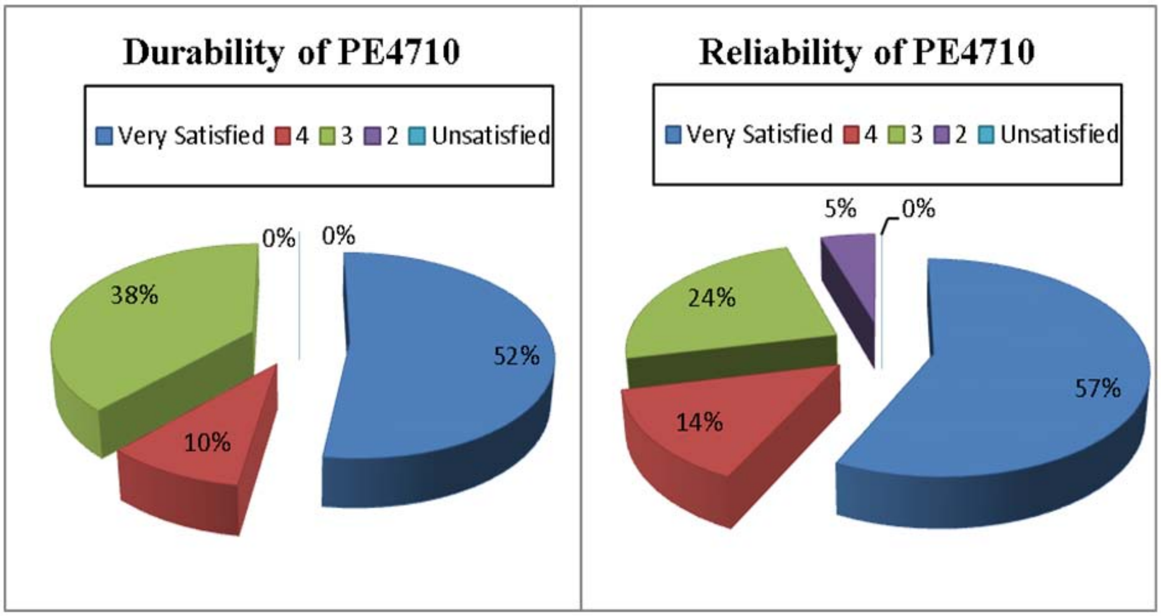

Figure 9. Percentage of respondents rating for *durability & reliability of* PE4710 (based on 21 respondents).

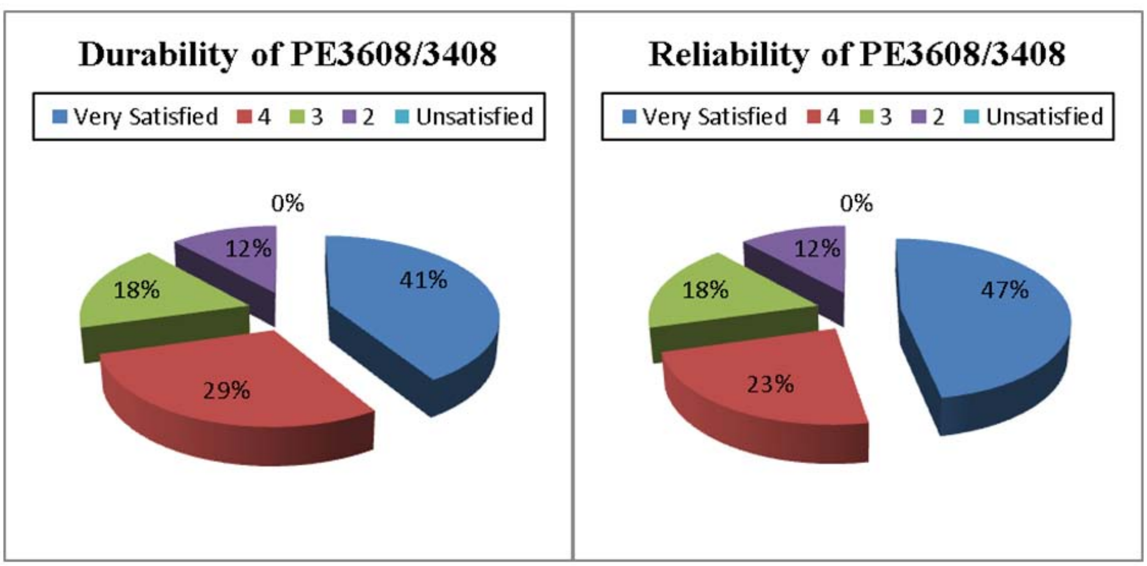

Figure 10. Percentage of respondents rating for durability & reliability of PE3608/3408 (based on 17 respondents).

Comments and Suggestions from Water Utilities

Table 1 summarizes general comments received from responding water utilities.

Table 1. General comment from responding water utilities.

Comments	Description
Leakage issues	1. Failure at manhole and service connections. 2. Leakages are found mainly at fittings, and flanged adapter to DIP joints. 3. Improper welding of joints. 4. Damage due to contractor's equipment.
General concerns	1. Molded fittings for pipes larger than 12-in. are not available, therefore fabricated fittings are the largest concern. 2. Additional permeation testing recommended especially at joints. 3. Problems in end caps, service connections, manhole connections and oxidation. 4. Accelerated testing is required to define the expected life of large diameters.
Positive comments	1. Water hammer/high pressures are major problems for C900 PVC, so HDPE was installed. 2. Suitable for area of landslides with high pressures.

CONCLUSIONS

Majority of responding water utilities, which had large diameter PE4710 pipe, were satisfied with its performance. They rated cracking, permeation and oxidation to be minor issues. Survey respondents expressed concerns about tapping, repairs, joints, and indicated measures are required to improve construction techniques, as described in this paper.

LIST OF ACRONYMS

ACP	Asbestos Concrete Pipe
CUIRE	Center for Underground Infrastructure Research and Education
DIP	Ductile Iron Pipe
HDPE	High Density Polyethylene Pipe
PCCP	Prestressed Concrete Cylinder Pipe
PPI	Plastic Pipe Institute
PVC	Polyvinyl Chloride
SP	Steel Pipe
TRWD	Tarrant Regional Water District
USEPA	United Sates Environmental Protection Agency

WaterRF	Water Research Foundation
WERF	Water Environment Research Foundation

ACKNOWLEDGEMENTS

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Can a Design Engineer Rely on D/t Ratio as a Rational Indicator to Manage Stresses and Strains in Welded Steel Pipe During Handling?

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Abstract

The choice of initial wall thickness in welded steel pipe design is accomplished by following an empirical formula with the objective of managing stresses and strains in the pipe wall during handling. Stresses and strains during handling are due to a number of factors, including lifting with slings, placement on supports, and shipping to name a few. Although the avoidance of cement mortar lining (CML) cracking before or during stull installation is of primary interest in this paper, the need to manage the strain level in steel to prevent having an adverse impact on the integrity of CML is also discussed. With a flexible lining, the design engineer has more latitude. To avoid excessive stresses and strains, for decades, design standards and manuals have provided empirical formulae as a diameter to thickness (D/t) ratio criterion. Given that water pipes carry relatively low internal pressures in many projects, the choice of initial wall thickness often ends up as the final thickness of the steel pipe. The question that has not been asked by engineers for over eight decades is "Can a Design Engineer Rely on D/t Ratio as a Rational Indicator to Manage Stresses and Strains in Welded Steel Pipe during Handling?" The authors of this paper embarked on an investigation to answer this very question. The authors share their methodology, results, findings and suggestions for improvement in current practice.

INTRODUCTION

When designing welded steel pipe, the initial minimum wall thickness "for handling" is selected. For decades, design standards and manuals provided empirical formulae as a diameter to thickness (D/t) ratio criterion for design engineers to meet of the forms:

Wall thickness, $t > (D+20)/400$ U.S. Bureau of Reclamation (BOR) for $D > 54$"
Wall thickness, $t > D/288$ Pacific Gas and Electric (PG&E) for D up to 54"
Wall thickness, $t > D/240$

AWWA (2004) incorrectly attributes these three formulae to Parmakian (1982), although he did not even mention the third one and demonstrated that neither of the first two formulae are supported by the theory of shells nor yield reliable guidance;

even decades later AWWA(2004) and AISI (2007), however, still include all three of the above formulae. The first two formulae have been in use for longer than 80 years. Both BOR and PG&E have been applying the above formulae for all pipe sizes. Other design criteria considered by the engineer include the pipe having sufficient hoop strength to withstand internal pressure, adequate buckling capacity, proper choice of bedding and backfill to maintain deflection less than a limiting value and the combined stresses from all loads not exceeding a certain percentage of minimum yield strength for the grade of steel plate chosen. The term “plate” is used herein to conform to the theory of plates and shells and not intended to claim that the steel pipes are fabricated exclusively from plates. Most water pipelines handle internal pressures that are sufficiently low such that the thickness required for internal pressure design is less than the thickness determined by these empirical “handling” formulae. Engineers from the ductile iron pipe research association, Horn and Breslin (2001), wrote about steel pipe “The design approach can result in a wall thickness calculation that leaves a pipe not stiff enough or without sufficient beam strength to stand alone during installation. In fact, handling considerations can potentially govern wall thickness design. One may go through the wall thickness design procedure and calculate a required wall thickness based on internal pressure and external load but find that the walls are still too thin to handle the pipe. Therefore, after accomplishing design, a check must be made to ensure that a minimum wall thickness (as a function of pipe diameter) is present.”

Given that these empirical formulae for D/t ratio often end up being the controlling selection criterion for design of steel pipe, the authors pondered the validity of this practice. This paper details an investigation by the authors to the above question, using closed form solutions for the range of welded steel pipes normally manufactured for use on water projects. The investigation studied 42 different sizes of welded steel pipe with the following lower and upper bounds as shown in the AISI-STI-SPFA Welded Steel Pipe Design Manual:

Pipe Diameter (inches): 4 to 156
Wall Thickness (inches): Ranging from 0.0747 to 0.1875 for 4-inch diameter; through 0.4375 to 1.00 for 156-inch diameter
D/t Ratio: 21 to 54 for 4-inch diameter; 156 to 357 for 156-inch diameter

To consider a statistically significant number of cases, the authors have included pipe sizes larger than 108 inches when in fact, it is rare to use factory applied cement mortar lining in these larger sizes.

HANDLING ISSUES

Due to continuity, the strain in the cement mortar is identical to the strain in the pipe wall at the interface between the mortar and the steel. The tensile capacity of steel at yield is ~900 microstrain and the capacity of cement mortar is ~125 microstrain. Consequently, strains in the steel pipe that exceed the tensile strain limit of cement mortar will result in cracking of the mortar lining. During manufacture of cement mortar lined steel pipe, stresses and strains must be managed in accordance with differing criteria for each step of the process. Subsequent to initial fabrication of the steel pipe cylinder, it must be able to withstand the stresses and strains of the self-weight of the steel cylinder. The pipe must also be able to withstand any stresses which may occur as the pipe is spun during the application of the cement mortar lining, taking into account the weight of the cement mortar lining itself. Once the mortar lining is sufficiently cured to allow the placement of internal bracing or stulls, the internal bracing will re-round the pipe from the deflected shape induced by its own self weight of the steel plate and the lining. Shipping, handling, and installation of the stulled pipe can induce stresses and strains that may also result in cracking of the cement mortar lining.

From experience, the water pipe industry and the highway drainage pipe industry have developed minimum and maximum recommended thicknesses as exemplified by the American Iron and Steel Institute (AISI) and the American Association of State Highway Transportation Officials (AASHTO), which are plotted in Figure 1.

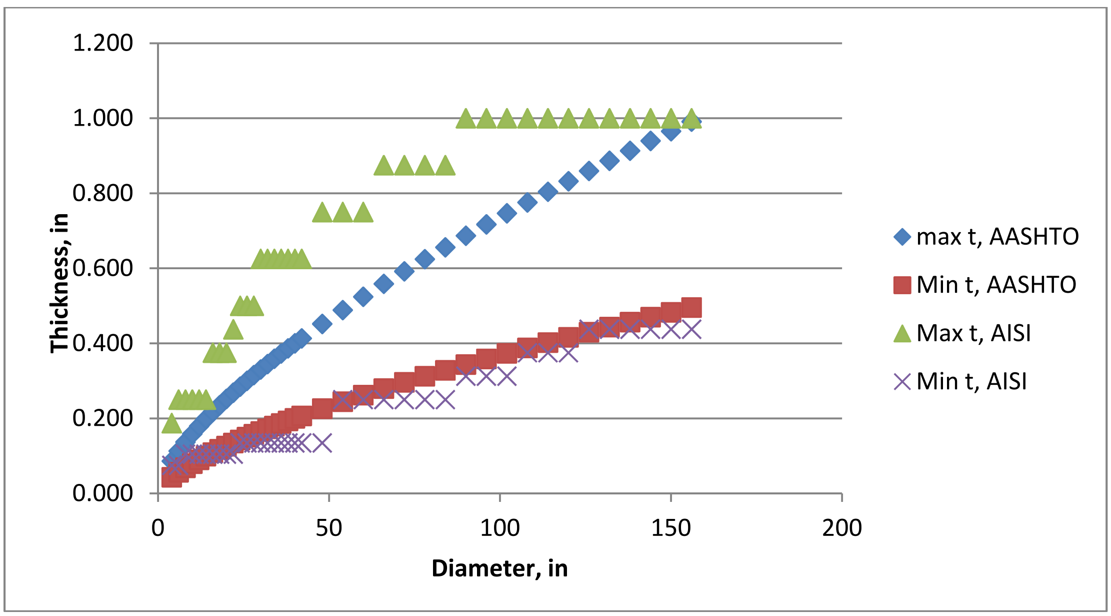

Figure 1. Minimum and maximum thickness per AISI and AASHTO

AISI minimum and maximum thickness values used in the potable water industry increase step-wise, based on standard thickness increments, whereas thickness for both smooth and corrugated wall pipes in the highway drainage market is governed by flexibility factor in accordance with AASHTO (2010) Section 12.5.6, which includes a wide range of values for the flexibility factor. To be able to keep the analyses for this paper manageable a representative range of 0.01 to 0.08 for steel pipes given in Roseke (2013) was used, which corresponds to minimum and maximum flexibility factors used for available corrugated wall pipes. Since pipe stiffness, and its reciprocal flexibility factor are dependent on the moment of inertia of the pipe wall, regardless of whether that moment of inertia is for the smooth-wall pipe or the corrugated-wall pipe, this range is applicable to all types of steel pipe.

The flexibility factor, FF is given by,

$$FF = D^2/EI, \quad (1)$$

Where, D is the diameter in inches, E is the modulus of elasticity of the pipe material in psi, and I is the moment of inertia of the pipe wall in $inch^4/inch$. The above steel plate thickness and diameter data as D/t ratio relative to diameter gives the relationship shown in Figure 2.

There are many interesting observations that can be made from Figure 2. There indeed is a linear relationship between D/t minimum in AISI and the pipe diameter. The coefficient of determination is 0.977 signaling that the confidence level of a linear correlation is remarkable. Unfortunately, these values are purely academic from a "handling" point of view, and for economical reasons - no designer would endeavor to recommend their clients choose wall thickness following this lower bound solution of D/t, unless warranted by other performance requirements. The values of D/t maximum according to AISI result in plate thicknesses that range from the thinnest steel plate per AASHTO to even less conservative with an upper bound D/t of 360 for pipe diameters 48-inches and greater. Due to the step-wise selection of standard thicknesses, the trend line is absent for the D/t max according to AISI. For both D/t min and D/t max, however, the AASHTO suggested D/t appears to have a one to one relationship of a higher order polynomial with D and the values result in designs in the middle of the extreme designs from D/t min and D/t max used by AISI.

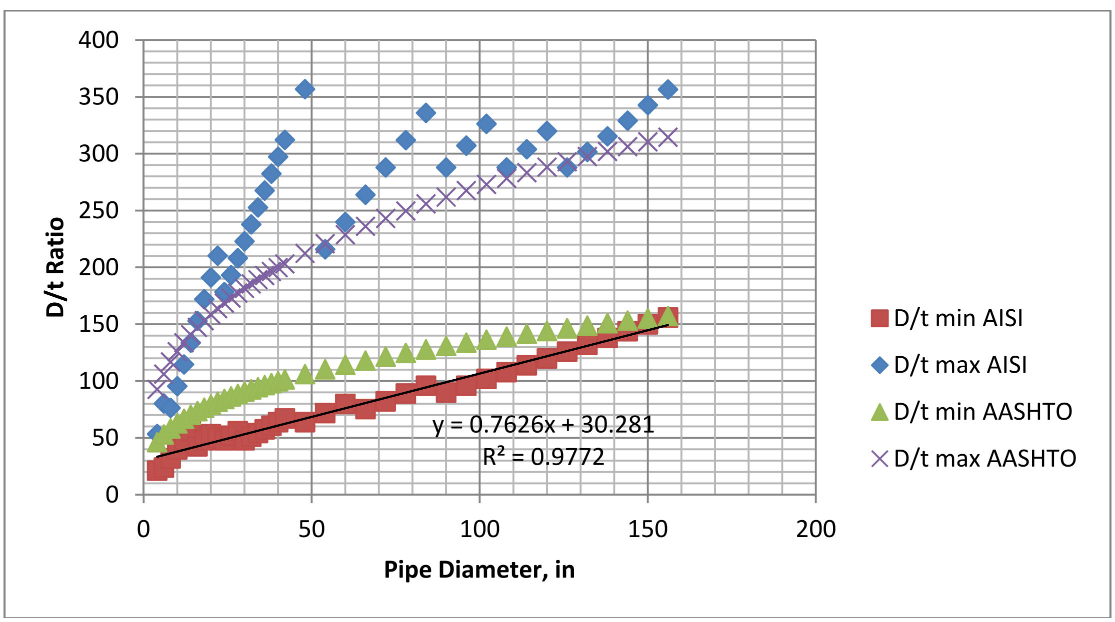

Figure 2. D/t Ratio as a function of pipe diameter

The authors feel that use of a continuous, mathematically selected boundary, such as the AASHTO data shown above, allows the design engineer to better understand the criteria governing handling issues when selecting appropriate pipe thickness, particularly if non-standard thicknesses are being considered.

STRAIN IN THE PIPE WALL

In order to control cracking of the cement mortar lining, strains in the pipe wall must be managed. Stresses and associated strains due to self-weight in straight beams of constant rectangular section are functions of the dimensional group, ϕ_1,

$$\phi_1 = D^2/t \quad (2)$$

Curved beams, such as that represented by the transverse section of a pipe wall, however, must take into account the effects of arching as well. The equations for the bending moments due to self-weight of a thin-walled cylinder supported at its invert shown in Figure 3, are from Roark and Young (1975) and this model would suffice given the pipe is well within linear elastic stress strain behavior for steel.

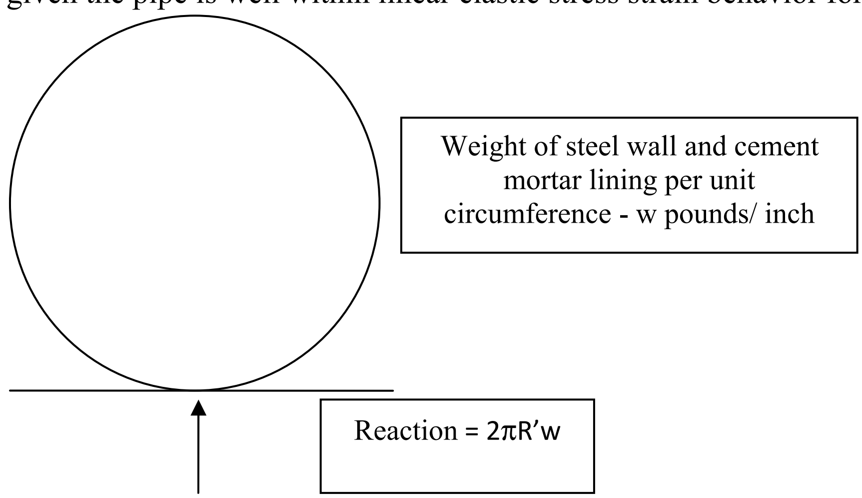

Figure 3. Thin walled hollow cylinder resting on a flat surface

Although good manufacturing practice does not support the pipe in this manner during the process of cement mortar lining, given that the primary objective of this paper is to demonstrate the comparative trends of pipe behavior with various parameters, the use of such a simplified model saves time and effort without introducing error in the comparative analysis. Based on the theory of similitude, even with a more sophisticated analysis using more realistic supports, for example belts or cradles, the bending moments still would be governed by variations of the form shown in equations 3 and 4 with the exception that the functions that multiply wR'^2 would result in values lower than those from the model which is chosen here for simplicity. A more accurate model, as shown in Figure 4 would result in equations 11 and 12 for bending moments at the crown and invert, respectively.

$M_C = wR'^2k_4/2$ (at the crown) (3)

$M_I = wR'^2(2-k_4/2)$ (at the invert) (4)

Where, w = the self-weight of the pipe wall per unit length, lbs/in and R' = the radius to the centroid of the pipe wall, in

$k_1 = 1 + \alpha + \beta$ (5)

$k_2 = 1 - \alpha + \beta$ (6)

$k_3 = 1 + \alpha - \beta$ (7)

$k_4 = k_2/k_1$ (8)

and,

$\alpha = I/(AR'^2)$ (9)

$\beta = FEI/(GAR'^2)$ (10)

I = moment of inertia of the pipe wall = $t^3/12$ per unit length of pipe, in^4/in

F = form factor for curved beams = 6/5 for the pipe wall's rectangular cross-section

E = modulus of elasticity of the pipe material, psi

G = shear modulus of the pipe material, psi

A = the cross-sectional area of the pipe wall = t per unit length of pipe, in^2/in

For this investigation, the following assumptions have been made to allow for the weight and the stiffness contributed to the steel plate by the presence of the cement mortar lining:

m = Modular ratio = 8; E = 30 Mpsi for steel, and 3.75 Mpsi for cement mortar

γ_m = unit weight of cement mortar = 3/10 the unit weight of steel

t_m = thickness of cement mortar per AWWA C205 (2012) as given in Table 1.

Table 1. Mortar Lining Thickness per AWWA C205

Nominal Diameter, in	Mortar Lining Thickness, in
4-10	1/4
11-23	5/16
24-36	3/8
Over 36	1/2

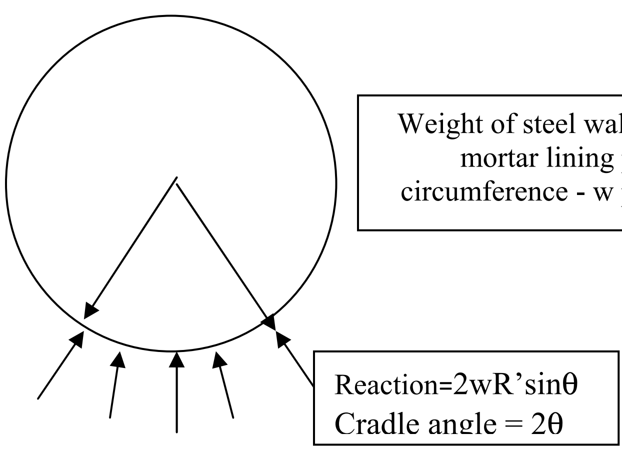

Figure 4. Thin walled hollow cylinder resting on a cradle

$$M_C = wR'^2\{2\sin\theta + \theta - \pi\cos\theta + \theta\cos\theta - \pi\}/\pi \qquad \text{(at the crown)} \qquad (11)$$

$$M_I = wR'^2\{-\theta\cos\theta + \theta\}/\pi \qquad \text{(at the invert)} \qquad (12)$$

Up to a D/t ratio of about 160, the strain in the pipe wall can be approximated by a straight line with a coefficient of determination, R^2 of 0.944, as shown in Figure 5. When D/t ratio exceeds 200, however, the ability to predict strain in the pipe wall using D/t ratio alone becomes weak, given that the coefficient of determination is only 0.439, as shown in Figure 6. Furthermore, it is evident that for D/t ratios in the 300 to 360 range, strain varies widely, and is in fact more a function of diameter than that of D/t. To further illustrate the non-linear relationship between D/t and strain in the pipe wall, the relationship between pipe diameter and tensile strain for a constant D/t = 288 is shown in Figure 7. The small steps in this curve are due to step-wise increases in the mortar lining thickness per AWWA (2012), as discussed above. Similar trends for D/t of 240 and 200 are shown in Figures 8 and 9. The corresponding maximum strains at the invert of the 156 in pipe for D/t values of 288, 240 and 200 are 393, 371 and 345 microstrains, respectively. The cradle model would likely yield only about 1/3 of these values under optimum uniform support of the pipe. The coefficients of determination when second order polynomials are chosen for the relationships between the strain in the pipe wall and the pipe diameter are 0.999, 0.999 and 0.998, respectively. The issue of stresses and strains in the pipe wall become more complex when longitudinal effects are considered, especially when the D/t value gets to be higher than 250 in small diameter pipes. Such longitudinal effects, however, are beyond the scope of this paper, and would be the subject for a paper in the future.

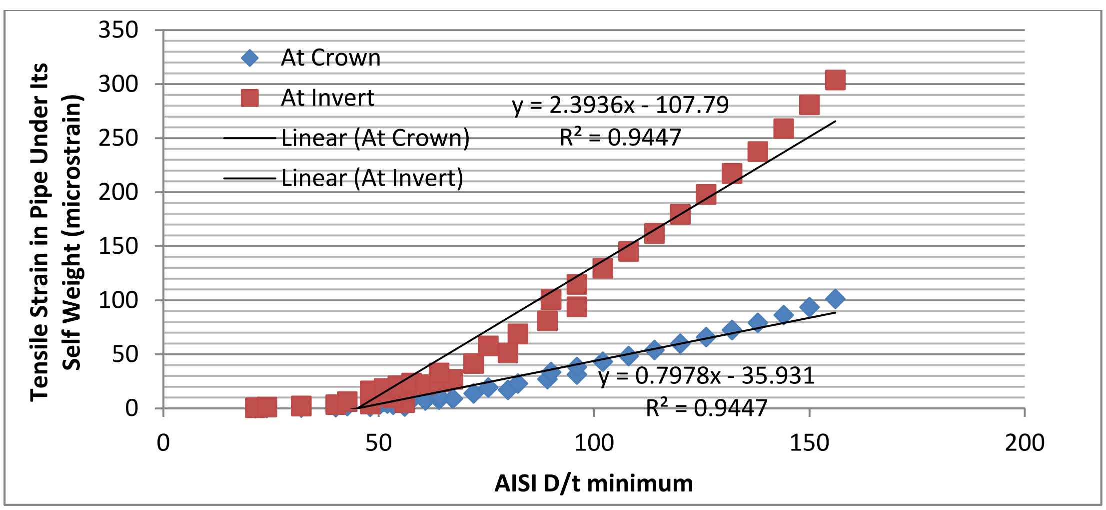

Figure 5. Relationship between D/t and strain in pipe wall for D/t < 160

MANUFACTURING PRACTICE

Experience and practices in the pipe fabricating mill and the field lend credence to the validity of the issues raised in this paper. For example, the likelihood of tension cracks occurring in cement mortar lining for pipes larger than 108 or 120 inches can be lowered by having the contractor line the pipe in the field once it is installed and backfilled, and the stulls are removed. Empirical knowledge, garnered over many years by doing the same task hundreds of times, allows pipe manufacturers to avoid using D/t ratios as selection criteria in these larger pipes. No such guidance is provided, however, to the design engineer using design standards such as AWWA M11 (2004). Accordingly, the use of D/t for controlling tensile strain in the pipe wall and the associated possibility of cracks in the mortar lining, when stulls are not used during handling is not supported by the underlying engineering principles. Consequently, the authors recommend that use of empirical formulas based on D/t should be abandoned in favor of more representative relationships.

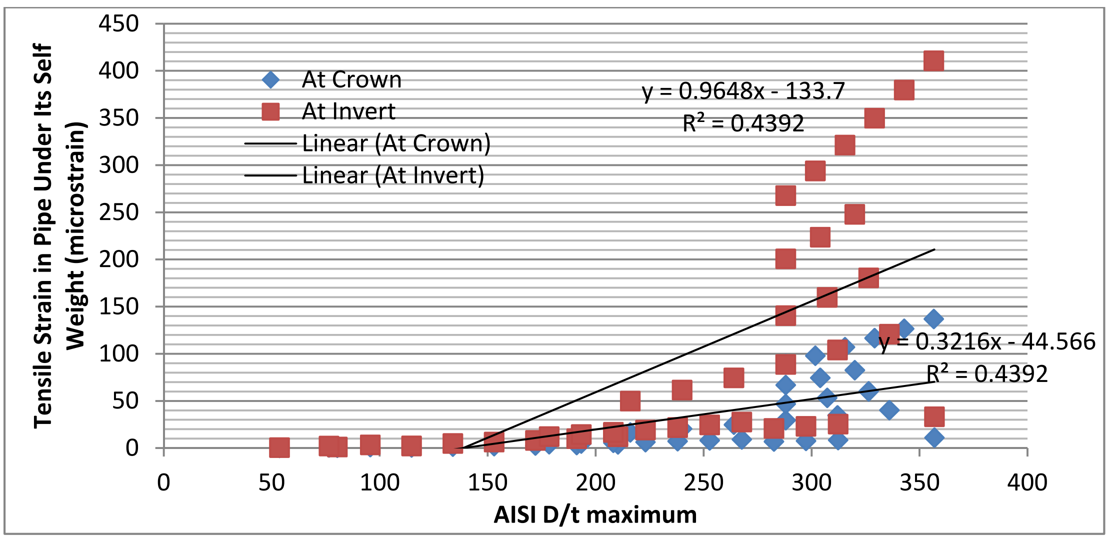

Figure 6. D/t and strain in pipe wall - poor linear correlation for D/t > 200

A MORE PRUDENT APPROACH

The values of maximum pressures a pipe can carry with the factor of safety of 2 for handling steel plate thickness using the empirical formulae are summarized in Table 2. Occasionally, a design engineer is called upon to cope with water pressures higher than these upper bounds, but most of the time pressures are below these limits. Furthermore, deflection and buckling are best controlled with backfill properties. Consequently, thickness selection has most often been governed by "handling" as determined by D/t ratio. Therefore, since the empirical formulae for D/t ratio are not reliable predictors of tensile strain in the pipe wall and consequently the mortar lining, a more accurate method should be adopted.

ACCEPTABLE CRACK SIZE IN MORTAR LINING

AWWA C205 (2012) acknowledges the process of autogenous healing of the cement mortar lining, by stating that cracks narrower than 1/16th of an inch (0.06 inch or 1.5

mm) need no repair. The most widely used criterion in European Standards for autogenous healing, however, is when the crack width is narrower than 0.01 inch (0.25 mm) with the presence of sufficient hydraulic pressure and adequate water, according to Edvarsen (1999) and Neville (2002). For example the crack width allowable from BS EN 10224 that applies to pipes 1 to 108 inches (26.9 to 2743 mm) in size reads, "Cracks up to 0.25 mm in width in saturated linings and not over 300 mm in length shall not be a cause for rejection." Although some level of cracking can be acceptable, it must be limited through properly predicting stresses and strains in the pipe wall. The failure of pipelines in Saudi Arabia as reported by Malik (1991) due to cracking in the cement mortar lining and the Metropolitan Water Districts losing the cement mortar lining over the length of 35% of its 5.3 mile 144 inch Etiwanda pipeline, by McReynolds et al. (2010), although this was lined in the field, emphasize the need to manage crack widths in cement mortar lining by managing the stresses and strains in welded steel pipelines. In fact, AWWA C205(2012) cautions "Consideration should be given to limiting the maximum strains (or stresses) in the steel cylinder of cement-mortar-lined or coated steel water pipe from internal pressure to ensure the long-term design life of the system." It should be emphasized that the management of strains in the steel wall and the CML during fabrication and handling is somewhat different from what is required during installation and service.

It appears that the best possible fit to the data for the relationship between strain and D/t ratio is of the exponential form as shown in Figure 10. The coefficient of determination improves from 0.439 to 0.822 but not as confident as the 0.944 for the linear relationship between the strain and the D/t ratio used in AISI minimum. It should be noted that the coefficient of determination is affected by the step-wise selection of standard thicknesses presented in the AISI data. Use of a rational equation for the lower bound of pipe thickness would result in a higher coefficient of determination.

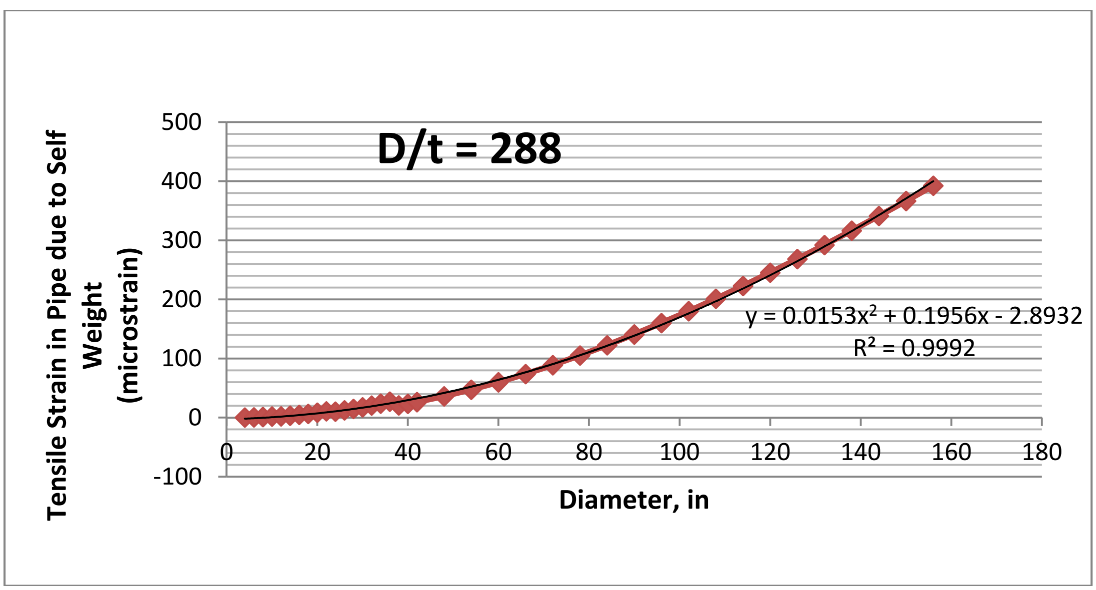

Figure 7. Strain in pipe wall and diameter for constant D/t

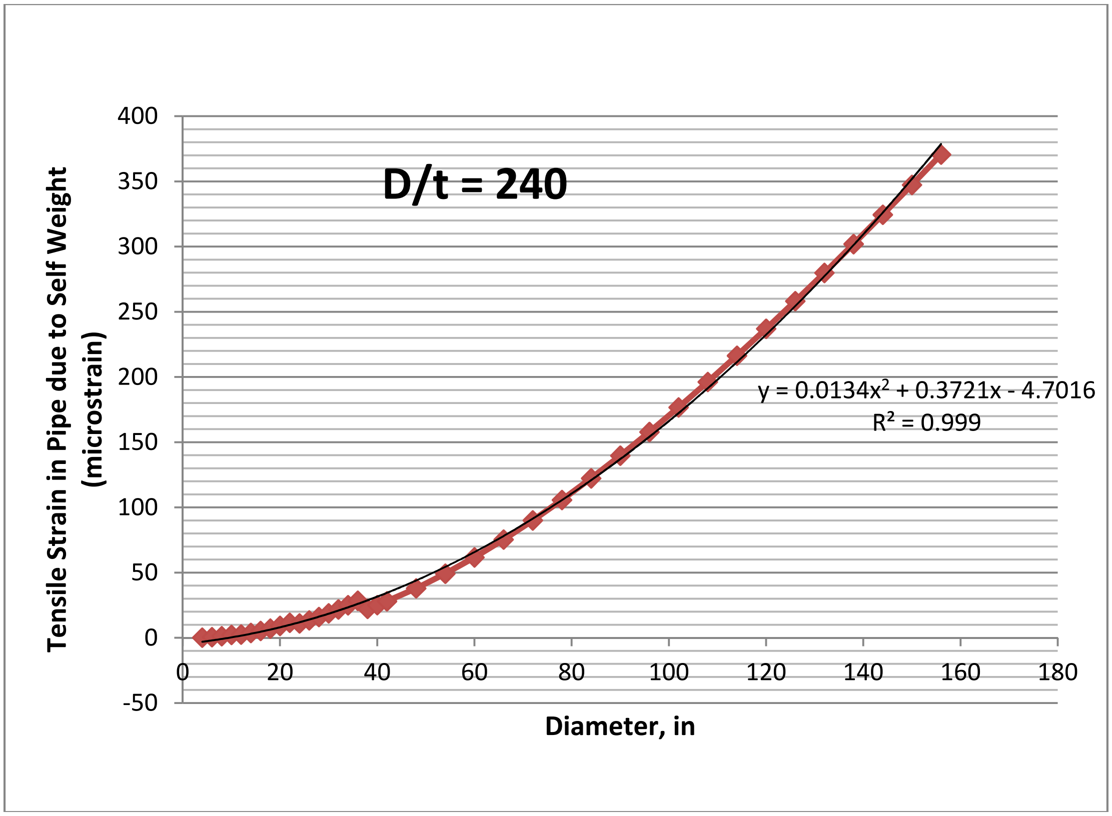

Figure 8. Strain in pipe wall and diameter for constant D/t

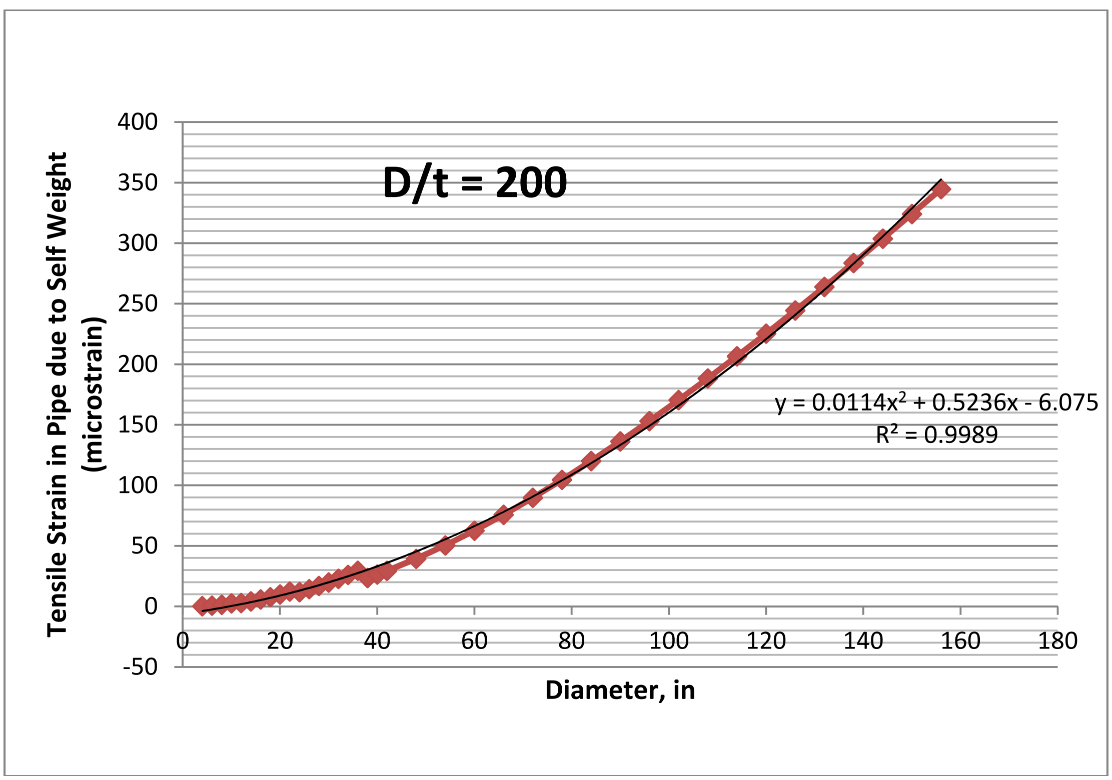

Figure 9. Strain in pipe wall and diameter for constant D/t

Table 2. Internal Pressures for Handling Steel Plate Thickness

D(in.)	(D+20)/400	D/288	Min(in.)	D/240	tmax(in.)	Pressure for fy = 36ksi	40ksi	44ksi
4	*	0.014	0.0747	0.0167	0.0747	672	747	822
6	*	0.021	0.0747	0.0250	0.0747	448	498	548
12	*	0.042	0.0747	0.0500	0.0747	224	249	274
24	*	0.083	0.0747	0.1000	0.1000	150	167	183
36	*	0.125	0.0747	0.1500	0.1500	150	167	183
60	0.2	*	0.0747	0.2500	0.2500	150	167	183
84	0.26	*	0.0747	0.3500	0.3500	150	167	183
120	0.35	*	0.0747	0.5000	0.5000	150	167	183
156	0.44	*	0.0747	0.6500	0.6500	150	167	183

*Not recommended per AWWA (2004)

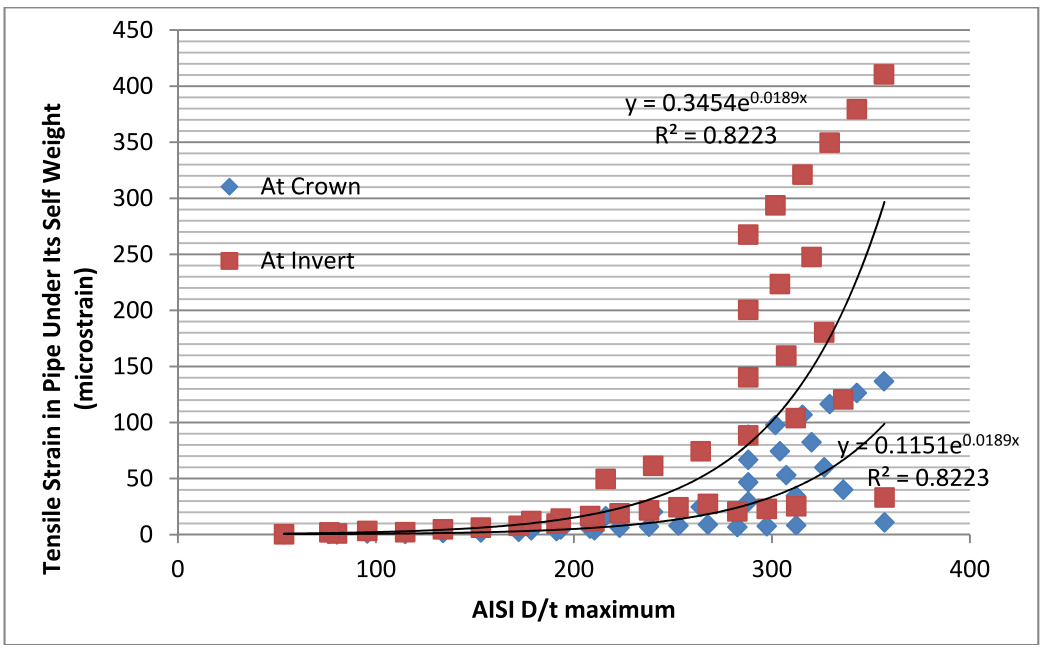

Figure 10. Better relationship for D/t and strain in pipe wall - D/t > 200

CONCLUSIONS

The primary conclusions are:

1) The engineering practice of relying on linear relationships between the D/t ratio and the strains in the pipe wall and consequently maximum strains in the cement mortar lining during handling of the steel pipe is not supported by the underlying mechanical principles. Fortunately, most manufacturers use round up rings during the lining process and stulls until the pipe is put in service.
2) Although the process of autogenous healing of cracks in the cement mortar lining allows some tension cracks to be acceptable, cracking needs to be limited through

the proper understanding and prediction of stresses and strains in the underlying steel pipe wall.

3) This inability of the practicing engineer to use D/t to predict stresses and strains in the pipe wall and also thereby manage tensile induced cracking in the cement mortar lining during handling should be recognized in the AWWA Manual of Practice M11. Therefore, rather than continuing to rely on the current D/t-based empirical formulae for choosing initial steel plate thickness and consequently choosing this initial thickness as the final design thickness, the use of these formulae should be discontinued in favor of more accurate representations.
4) A better approach to managing stresses and strains in the steel pipe wall is to develop a relationship between steel plate thickness and diameter, which keeps stresses and strains in the pipe wall below an acceptable maximum, supported by a rigorous statistical evaluation.

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Stulling of Large Diameter Steel Water Pipe—What It Is and What It Is Not

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Abstract

Large diameter steel water pipe is typically supplied with temporary internal bracing or "stulls" that have been shown to prevent damage to the pipe, particularly rigid lining and coating systems such as cement mortar lining and coating, during transportation and handling. They are also provided to assist with maintaining the shape of pipe ends to within AWWA C200 requirements for field jointing. Stulls are typically manufactured from rough cut lumber and on occasion, steel angle or poles. Stull sets are typically placed at pipe ends and at mid points, depending on diameter and thickness of the pipe. Stull sets typically consist of a minimum of one to three sets of stulls with various stiffeners, and may include shoes and blocks to assure the internal bracing will retain functionality during the handling process. Manufacturers may have variations in their stulling means and methods but all have the same goal of providing pipe integrity during handling and transport, and pipe-end roundness at the ditch, ready for joint assembly, installation and backfilling. Stulls may assist holding the pipe shape until side fill support is developed during backfill placement. Loads on top of the pipe, including construction loads, are distributed to the soil envelope around and adjacent to the pipe. The assumption that pipe stulls will limit or eliminate deflection has lead to misunderstandings, false expectations and even improper pipe installation due to over reliance on a stulling system's ability to limit deflection of the pipe. Pipe stulls are not in themselves designed to withstand the unknown loadings generated by depth of soil cover or various construction equipment and vehicles during installation. This paper will illustrate the installation of stull sets at the factory, review their intended function, discuss flexible pipe deflection control methods and review installation requirements of the AWWA C604 buried steel pipe installation standard.

INTRODUCTION

The purpose of a stull is to maintain the integrity of the pipe cylinder, lining and coating during handling and transport. Manufacturers use the inherent pipe stiffness, Diameter-to-Thickness ratio (D/t), combined with the lining and coating requirements and their knowledge of the equipment that will be handling the pipe to determine the need, configuration and location of stulls. Pipe stulls are not in themselves designed to withstand the unknown loadings generated by various construction equipment and vehicles during installation.

The basis of stull layouts and configurations are empirical. They are not typically based on a pure mathematical design, but rather the successful historical use and experience of the steel pipe industry. There is not one set method that can account for the various handling and transportation means that may be found in different manufacturing facilities.

Guidance has been provided in the Steel Penstock Manual (ASCE 2012), Chapter 13, which includes a table that establishes criteria for wood stulling of pipe with nominal diameter up to 120-in, Table 1. This table may be modified by the manufacturer to facilitate the proper protection of the pipe and its linings and coatings for handling and transportation.

Table 1: Wood Stull Criteria

	Pipe diameter/Stull size					
	D = <24 in.	D = 24 in. to <30 in.	D = 30 to <48 in.	D = 48 in. to <60 in.	D = 60 in. to <84 in.	D = 84 in. to 120 in. Cement-mortar lined pipe only
Diameter to thickness ratio (D/t)	2 in. × 6 in.	3 in. × 3 in.	3 in. × 3 in. for 30 in. and 4 in. × 4 in. for larger diameters	4 in. × 4 in.	4 in. × 4 in.	4 in. × 4 in.
$D/t \le 120$	No stulls	No stulls	No stulls	No stulls	No stulls	No stulls
$120 < D/t \le 160$	Brace between bunks	2 stulls vertical	2 stulls crossed	2 stulls crossed	2 stulls, 3 legs	2 stulls, 3 legs
$160 < D/t \le 200$	Brace between bunks	2 stulls vertical	2 stulls crossed	2 stulls, 3 legs	3 stulls, 3 legs	3 stulls, 3 legs
$200 < D/t \le 230$	—	2 stulls vertical	2 stulls crossed	3 stulls, 3 legs	3 stulls, 3 legs	3 stulls, 3 legs
$230 < D/t \le 288$	—	—	2 stulls, 3 legs	3 stulls, 3 legs	3 stulls, 3 legs	3 stulls, 3 legs

Notes: D = nominal pipe diameter; t = pipe wall thickness; stulls should be placed 15 to 20% of the total pipe length from each end, but no less than 4 ft in from end; and shipping bunks are to be located near stulls.

Table reproduced from ASCE (2012), Chapter 13

TYPES OF STULLS

Once the determination is made to include stulls, or stull sets inside the finished pipe cylinder, the next step is to determine the configuration. Often only one stull is required, and as such, the single stull is placed vertically, Figures 1a and 1b.

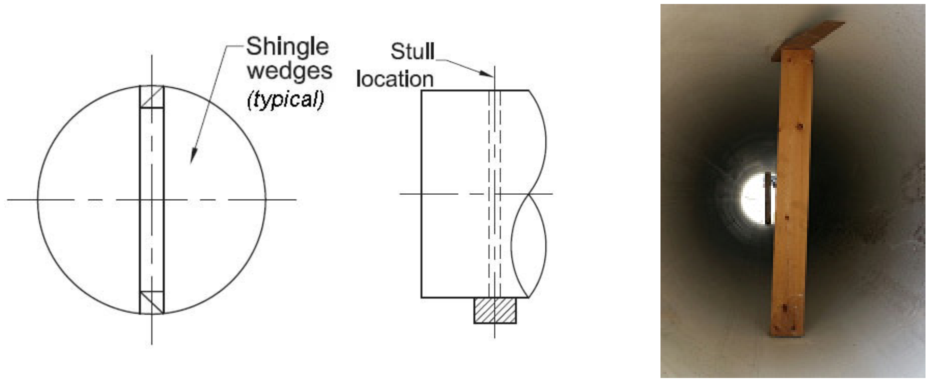

Figure 1a, b: Single Vertical Stull

If a 2-stull set is required based on the D/t ratio, a cross configuration is used to provide support in both vertical and horizontal axes. In the 2-stull configuration, one stull is vertical and the other is horizontal, Figures 2a and 2b.

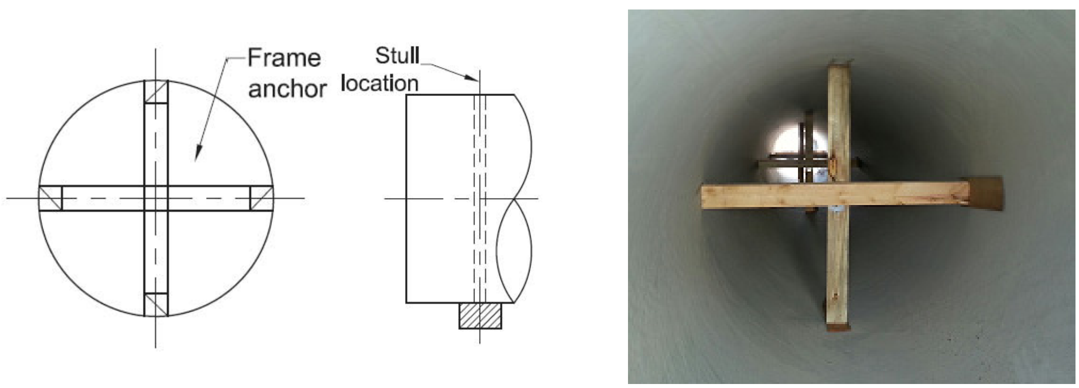

Figure 2a, b: 2-Stull Cross

For pipe cylinders with lower stiffness or a high D/t ratio, 3-stull sets oriented in a 60° configuration, commonly called a spider, are often used. In some cases, the vertical stull may be larger than the other 2 stulls in the set, Figures 3a and 3b.

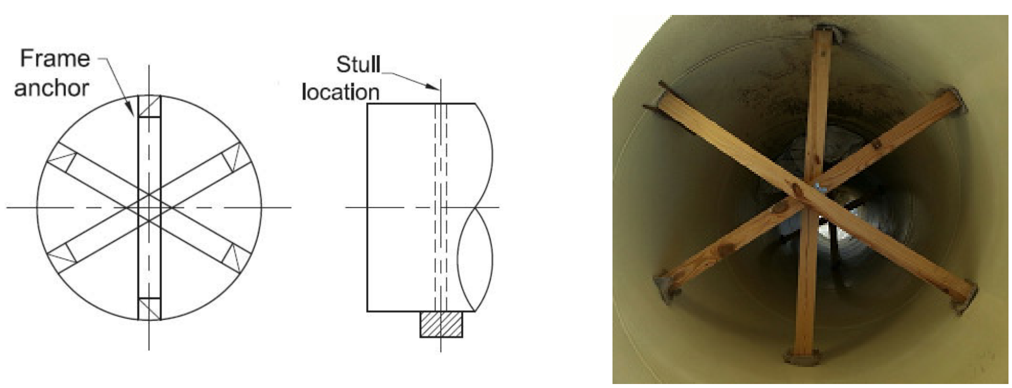

Figure 3a, b: 3-Stull 60° Spider

In rare circumstances small diameter steel tubing or a steel stull assembly, designed specifically for the application, is used to mitigate the possibility of excessive deflections during transportation and handling. This practice is typically limited to very large diameter, unlined pipe. The steel stull is often small diameter tubing that is inserted into a larger diameter "sleeve" welded to the inside of the cylinder. This type of stull set would need to be removed prior to the in situ lining operation. Additional guidance for steel stulls is available in the Steel Penstock Manual (ASCE (2012), Chapter 13

STULL LOCATIONS

To be effective, the proper number and location of stull sets should be used. The stull is the piece or pieces of material shown above (either single, double, 60° spider or special steel assembly) and the number of stulls is how the quantity and placement of the stull assembly is defined. At a minimum a stull set should align with storage and transportation bunks, and depending on the pipe stiffness and diameter, stulls will be placed in the center of the cylinder and near the pipe ends. Figure 4 shows 66-in pipe with 3 sets of stulls in a 3-stull 60° spider. In this case the pipe would be loaded for shipment with the bunks under the 2 outer stull sets.

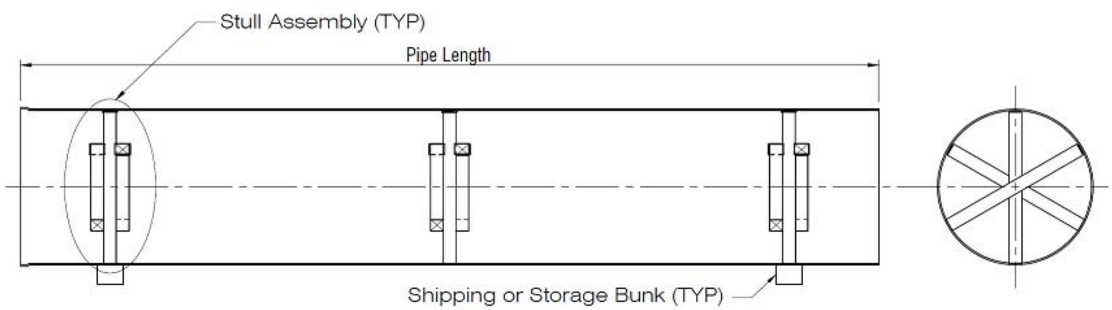

Figure 4: Completed Stull Set

STULL INSTALLATION

The placement of stulls is a relatively simple process. Figures 5 thru 14 show the installation of a stull set for a 66-in ID steel pipe with polyurethane lining and coating. This particular joint, when complete, will have 4 sets of stulls with 3 legs each. The lumber is first cut to length, Figure 5, typically just shorter than the inside diameter of the finished cylinder with the lining in place. Once the location is identified on the inside of the pipe, Figure 6, the vertical stull is fitted into place, often with a rubber mallet or small sledgehammer to ensure the lining is not damaged, Figure 7. Sometimes wood wedges or shims are used to adjust the length of the stull and to ensure the stull leg is firmly in place and will not fall out on its own if the pipe were to flex upon movement. For polyurethane or epoxy lined pipe, carpet is used on the ends of the stulls to protect the lining during shipping.

The subsequent legs are then fitted into place, Figures 8 and 9, again with firm contact to the inside of the pipe but not so roughly placed as to adversely affect the shape of the cylinder or to damage the lining in any way. In some cases, these stulls are then connected at their centers with wood screws or using a piece of angle iron and nails, Figure 12. Subsequent stull sets would then be placed inside the pipe in the same manner, keeping the vertical and horizontal orientation consistent throughout, Figures 11 and 13.

Figure 5: A pallet of stulls ready for use

Figure 6: The location of the inside stull is determined

Figure 7: The Vertical stull is fit into place

Figure 8: 2nd Leg is placed 60° from vertical

Figure 9: The final leg is placed, supporting the full circumference of the cylinder

Figure 10: Two inside stull sets are visible

Figure 11: All four sets, with consistent orientation throughout the cylinder

Figure 12: Angle and nails to connect the stull legs

Figure 13: Completed assembly down the length of the cylinder

DESIGN LIMITATIONS

Some have assumed that the standard stulling provided is there to limit any possible pipe deflection that can occur during backfilling. AWWA C604 – *Installation of Buried Steel Water Pipe* (AWWA 2011) details the fact that internal bracing (stulls) are provided for shipping and handling purposes “if required.” C604 also states “This bracing may or may not be adequate to limit pipe deflection during backfilling operations.” The reason for this caveat is that contractors have different means and methods regarding backfilling of pipe. The project specification may dictate type of material and/or level of compaction, but experienced contractors may have different approaches to satisfy these requirements. These subtle differences may have dramatic effects on the external loading of the cylinder during installation, thereby making it extremely difficult to design a stull set to account for these variations. Pipe stulling systems are not in themselves typically *designed* to withstand the unknown loadings generated by various types of construction equipment and vehicles during installation. This again stresses the importance of proper side soil support, and taking advantage of the pipe-soil interaction to maintain pipe shape during installation, backfill, construction and completion of the pipeline. A detailed discussion on the topic of pipe-soil interaction for buried flexible steel pipe can be found in Watkins et al. (2010).

In flexible pipe design, soil stiffness, not pipe stiffness, is the driving design consideration as the stiffness and strength of the compacted trench fill material essentially carries the live and dead loads of the pipe and prevents the flexible pipe from excessive deflection. Typically, the relative contribution of the soil stiffness to the resistance to allowable vertical deflection in buried flexible pipe is 97% while the pipe stiffness is only 3% (ASCE 2009). Stulls are provided to protect the integrity of the linings and coatings by limiting the flexibility of the cylinder during handling, transportation and pipe jointing.

Assuming that stulls will add significant pipe stiffness that will offset dead or live loads is problematic. As long as proper backfill techniques are followed per the AWWA C604 (2011) standard, the need for stulls to properly backfill flexible pipe is most often not warranted. The over reliance or misunderstandings on stulls “to keep the pipe round during backfill” can contribute to improperly installed flexible pipe. Key to proper installation of flexible pipes includes controlled lift techniques, proper compaction methods, balanced loading of backfill, and even monitoring of the horizontal and vertical movement of the pipe during the backfill process.

With proper dimensional monitoring, flexible pipe is good at telling an installer if the process being used is adequate. Deflection limits are defined by AWWA Manual M11 (2004) and/or project specifications and should be the guide to installation.

SUCCESSES

Proper placement of stull type, configuration, quantity and location can mitigate damage to the lining, coating and/or finished cylinder, thereby reducing costly and time consuming field repairs. They are also a useful aid in keeping the pipe joints within acceptable tolerance for joining adjacent cylinders in the trench, again saving installation time and costly fit-up expenses.

Figure 14 shows 108-in pipe being transported on padded forks. Figure 15 shows the inside of that same pipe joint, just prior to the end cap being installed. Bolts are used in this particular set but simply nailing the centers together has also proven effective. Figure 16 is a close up of the feet used for this particular stull configuration. The stull feet aren’t always necessary but they do help ensure proper sizing of the stulls and create a larger bearing surface on the inside of the pipe joint for increased stability.

Figure 14: 108” pipe on padded forks in manufacturing facility

Figure 15: 5 sets of a 3-stull spider configuration

Figure 16: Support feet at the end of each stull

Figure 17 shows a 66-in pipe joint as it is being placed in the trench. In this photo the stull can be seen near the end of the pipe, used to keep the joint within applicable tolerances.

Figure 17: 66-in pipe joint being placed in the trench

CONCLUSION

Regardless of the type of external loading, stulling is successfully used to prevent excessive cylinder deflection and particularly to prevent rigid lining or coating damage during transportation and handling. Pipe stulling systems are not in themselves designed to withstand the unknown loadings generated by depth of soil cover or various construction equipment and vehicles during installation. Properly installed flexible pipe relies on the pipe-soil interaction to maintain its shape prior to the completion of backfill operations. With adequate side soil support provided by proper backfill, this pipe-soil interaction allows for transfer of the load, preventing excessive pipe deflection. Once backfill is placed and compacted to a level to provide side support, some stulling may be removed to facilitate access for inspection and/or joint completion.

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Extremely Controlled Rock Blasting Near Critical Pipes Where Mechanical Excavation Is Not Practical

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Abstract

There are cases where excavations in hard rock must occur very close to existing pipes that must remain in operation. These cases include new gas or water pipes in existing Rights-of-way(ROW) where separation between new excavation trenches and existing pipes is very limited. In other instances, hard rock or mass concrete must be excavated to install new pipes or other facility upgrades near critical utility pipes or operating penstocks at hydroelectric plants. In these cases where excavations in hard rock must occur very close to existing pipes, the use of blasting methods is not possible when standard vibration criteria or restrictions based on pseudo-theoretical calculations are applied to the work. However, mechanical methods are often impractical when the rock is too hard, volume of rock is too large, or excavation geometries prevent their use. This paper focuses on systematic methods that can be used to develop customized blasting programs based on existing pipe(s) condition, strain failure modes, ground characteristics, and close-in blasting experience. The application of these methods is demonstrated in two case-history summaries including blasted rock excavations located within 1m of steel penstock pipes and 2.4m from steel water mains.

INTRODUCTION

Demand for infrastructure increases with population growth. A finding in a major study about water main break rates (Folkman, 2012), indicates about 264 people are served by each mile of water-main-pipe. Projections by the Pew Research Center estimate the current (YR 2014) population of 320 million people in the US will increase to 438 million by year 2050. A population expanding by 138 million people will require an additional 522,727 miles ($138 \times 10^6 /264$) of new water mains. Due to the environmental favorability and increased production, natural gas is rapidly replacing coal for electric power generation. The Kiplinger Letter (2015), reports that pipeline builders will be busy for years while adding enough new pipes to provide 20 billion cubic feet more of gas each day by Year 2020. This is more than a 25% increase in demand.

In the next 20 years or so, thousands of miles of new pipes will be installed and many will be in existing right-of-ways (ROWs) containing one or more operating pipes.

The use of mechanical methods or expansive grouts placed in drilled holes is not practical when large volumes of hard rock must be excavated near existing pipes. In cases where drill-blast methods are the best alternative for excavation work occurring within 3m of pipes, their use becomes extremely difficult or totally impractical when standard vibration limits or restrictions based on pseudo-theoretical calculations are applied to the work. These limits include the standard 125mm/s peak particle velocity limit recommended by the US Bureau of Mines (Siskind et al, 1993) and pipe stress predictions and limitations developed by the Southwest Research Institute (Esparza et al, 1981 and Esparza, 1991).

As more pipes are crowded into ground near existing pipes, the cost of the work is inflated due to unnecessarily high drilling and blasting costs when overly-restrictive limitations are applied. For excavations done very close to pipes, the work becomes impossible when traditional limitations are applied. For these challenges, project owners, government agencies and engineers will need to find other solutions.

Alternatively, as demonstrated by the blasting control procedures and case histories related in this review, careful blasting programs can be designed for projects where traditional blasting limitations are not practical. Presented methods focus on how to: 1) investigate specific site conditions, 2) identify real risks to pipes and other structures and, 3) develop appropriate controls and measurements to assure the work is successful.

BLAST EFFECTS

Providing a complete review of drilling and blasting terms is beyond the scope of this paper. It is presumed that readers interested in very close-in blasting will already have a basic understanding of general drill-blast methods. A major focus of this review concerns misunderstandings about the effects of blast-induced vibrations. When blasting is done carefully by competent blasters and overseen by experienced engineers, vibration alone will not damage buried pipes. Improperly controlled blasting that causes ground rupture or block movement of rock around pipes can cause damage. A brief review of blast effects follows.

When explosive charges detonate in rock, they are generally designed so most energy is used in breaking and displacing the rock mass. However, some energy will be released in the form of transient stress waves, which in turn cause temporary ground vibration. Detonating charges also create rock movement and release of high-pressure gas, which in turn induce air-overpressure waves (audible and inaudible noise), and airborne dust.

Direct and Permanent Rock Damage. Rock near heavily-charged-holes is often crushed or compressed and permanently damaged. The extent of this compressive and shear failure zone is usually limited to one or two charge radii. Beyond the crushing zone, rock or ground is temporarily deformed by elastic strain waves. For some distance, strain tangential to primary compression waves exceeds the rock's strength and new fractures are created. High pressure gas also contributes to the formation of radial cracks. The magnitude of dynamic strain and accompanying particle motion dissipate with distance. For fully-charged holes, radial cracks can extend up to 13 charge-diameters in competent rock. Damage and cracking of remaining rock around charges can be drastically reduced by using very light charges that only partially fill blast holes. Use of charges that are decoupled in this way is an important tool for preventing rupturing of ground around pipes.

Direct rupturing or overbreak of rock beyond the desired limits of a blast area might also occur if ground is weak or jointed and/or poor perimeter control methods are used for blasting. This important concern is highlighted in case histories and risk assessment examples covered later in this review.

Blast-Induced Vibration Waves. In ground that is not ruptured or displaced, detonating charges create stress waves that spread through ground and along open ground surfaces. Compression and shear waves pass through the "body" of the ground and Rayleigh waves travel along ground surface. These vibration waves travel at varying speeds and they are reflected, refracted and attenuated by various geological, structural, and topographical conditions. When blasting with many geometrically separated charges fired at varying delay times, it becomes virtually impossible to accurately model the exact motion-causing impacts of various strain waves.

Due to the extreme complexity of vibration-causing waves, empirical methods based on principles of dimensional similitude are generally used to predict and characterize blast-induced ground motion. Passing seismic waves cause ground to oscillate within three-dimensional space. Within a fraction of a second after blasting has stopped, vibration energy is dampened and ground particles become still.

Damage criteria are generally based on particle velocity and frequency of motion. The speed of strain waves in ground occur in 1,000s of meters per second but the resulting speed of motion they create in ground particles is quite low. Hence, the velocity of shaking ground particles is expressed in units of millimeters per second (mm/s) or inches per second (in/s).

The physical nature of vibrations caused by relatively small charges used in construction blasts is extremely different than ground shaking caused by earthquakes and by large-scale blasting at surface mines. Low-frequency lurching motions caused by moderate and severe earthquakes can rupture ground and damage buried pipes. With properly controlled close-in blasting work, the resulting high-frequency vibration will generate very low displacements and will not rupture ground or damage pipes.

Understanding the differences between high-frequency vibration and the slow-acting lurching motions that can cause damage to pipes and other structures is a key focus of this review. There are extreme differences between the displacements and corresponding ground strains caused by the lurching motion of earthquakes versus carefully designed blasts. For comparison purposes, in Figure 1, same-scale differences of ground vibration measured during the 0.17g Loma Prieta earthquake of 1989 is compared to blast-induced vibration measured on a concrete floor slab near various pipes within the Narrows 2 Power Plant in California. Rock blasting was done to facilitate construction of a new penstock bypass tunnel located within five feet of the plant in year 2006.

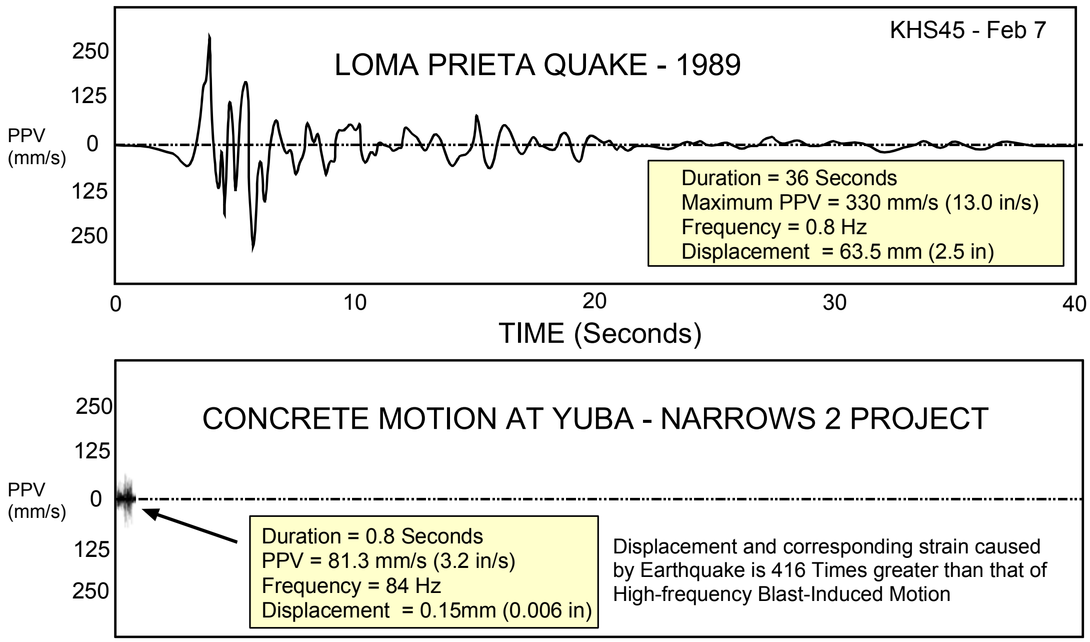


Figure 1. Scaled Comparison of Earthquake and Close-In Blast-Induced Motion

The PPV of the Narrows 2 blast at 81.3 mm/s with a frequency of motion at 84 Hz caused a temporary particle displacement of 0.15 mm, which is less than the thickness of a human hair. The Loma Prieta quake caused motion of 63.5 mm (2.5 in) and the shaking lasted 36 seconds, versus a second or two for a typical rock blast.

With high frequency motion, vibrating particles of ground are changing direction so quickly they are almost running in place. Imagine striking a steel tuning fork; the tines of an A-note fork shake back and forth 440 times a second. Your eyes barely see any motion but the peak particle velocity (PPV) in the forks can easily exceed 250 mm/s. Due to high frequency motion, the actual movement is tiny.

For the sinusoidal motions caused by blasting, particle displacement will generally equal PPV / ($2\pi f$), where f is the frequency of motion in Hz. Displacement is inversely proportional to frequency, so for any given PPV, displacement is reduced as frequency of motion increases.

It is also important to understand that vibrating particles of ground or components of pipes are not separated by the amount of particle displacement because, like two dancers on a ballroom floor, they are moving together just slightly out of step. For example, where the maximum elastic movement of concrete particles at the Yuba 32 Narrows plant was 0.15 mm, the actual separating strain between the particles of concrete is orders of magnitude less than the peak displacement. Due to this condition there is not enough differential shearing or tensile displacement to cause any damage or separation.

When evaluating the damage potential of vibratory motion, engineers and regulators should focus on the intensity of strains caused by flexural bending or displacement. The proximity of pipes, degree of confinement, and physical condition should also be considered in these evaluations.

MEASURING BLAST-INDUCED GROUND MOTION IN THE NEAR-FIELD

Our industry puts far too much focus on peak particle velocity (PPV) limits. For very close-in blasting, conventional seismographs using magnet-in-coil velocity transducers are often used incorrectly. When blasting hard rock within 3m of pipes or other structures, motions can have frequencies exceeding 1,000 Hz and particle velocities exceeding 1,000mm/s. In some conditions, slower-acting ground pressures created by a blast can cause enough ground movement to break pipes. This motion, similar to lurching earthquakes, will often occur at frequencies below 1Hz. The accurate frequency range of standard velocity transducers is generally between 2 to 250Hz. As demonstrated in the following case histories, accelerometers can be used in lieu of velocity transducers to obtain accurate near-field measurements. Conventional compliance seismographs using velocity transducers can also be used if the sensors are mounted at appropriate locations where the range of motion is within their measurement range. In cases where measurements at a point of concern - like a buried pipe - are not practical, measurements can be done at accessible locations and extrapolation methods can be used to predict worst-case motions at other points of concern.

EVALUATING RISK TO PIPES

Since pipes in operable condition are generally much stronger than surrounding soil, understanding the response of soil and rock is more important than evaluating the theoretical strength of the pipe. The key goal is to assure that no permanent ground displacement or block movement of rock occurs near the pipes. If charges are properly designed and placed so all motion near pipes is elastic and at higher frequencies, permanent ground rupture and pipe damage can be prevented.

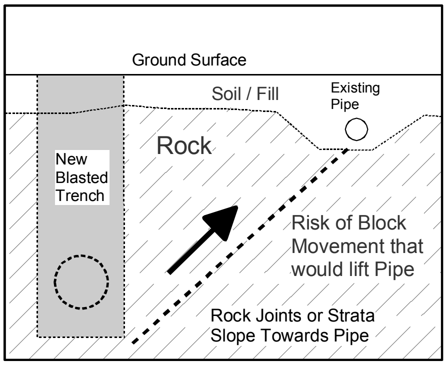
The potential for rupturing of the intervening ground (pillar) between two trenches is very dependent on the respective geometry of the trenches, trench separation, and the structural condition of the rock. Rock strength influences blasting but structural conditions like joints, weak bedding plane laminations, and shear zones will likely have more influence on potential for ground failures. The orientation of rock weaknesses with respect to the pipe and primary direction of expected blast heaving forces is also critically important.

For demonstration purposes, four simplified cases where ground conditions and excavation geometry could harm pipes are shown in Figure 2. These simplified cases are based on actual conditions the author has encountered over the last 30 years.

In practice, when high-risk blasting is planned in conditions like those shown in Cases 1 to 4, the nature of the ground between the blast area and pipes should be carefully investigated. Ground characterization can be determined by mapping rock outcrops, core drilling, seismic refraction tests, test pits, and other methods.

After characterizing ground conditions, potential ways that blasting might cause ground shifting or block movement which could damage pipes can be identified. Then measures like additional ground support, excavation geometry changes, and special blasting controls can be applied to prevent failures.

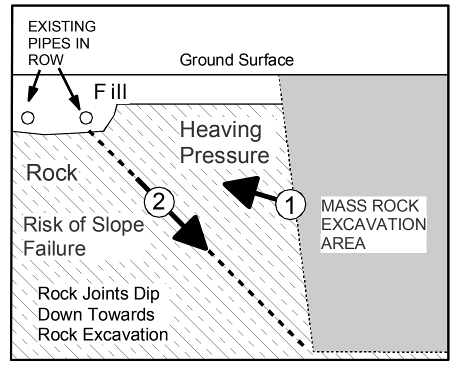
CASE 1 - GROUND UPLIFT RUPTURE



Ground and geometry conditions indicate there is risk that uplifted ground from blasting could damage an adjacent pipe.

Consider:
1) Increasing distance between the new trench and pipe.
2) Use open line-drilled or slot-drilled holes in the wall of the trench closest to the pipe.
3) Use small delay-decked charges to sequentially lift and move ground.

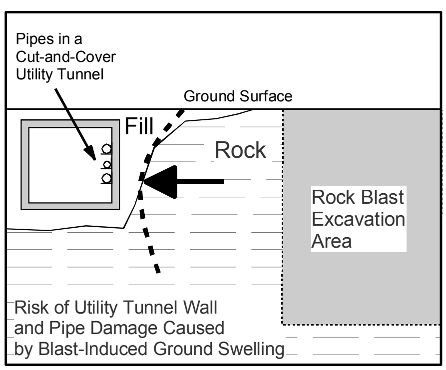
CASE 2 - SLOPE FAILURE



Ground and geometry conditions indicate the excavated slope could fail which would undercut and damage adjacent pipes.

Consider:
1) Installing vertical rock anchors in the the rock slope before blasting.
2) Blast open cut area with a series of short benches to reduce lateral force.
3) Use light charges in trim blasts with reduced widths against the final wall.

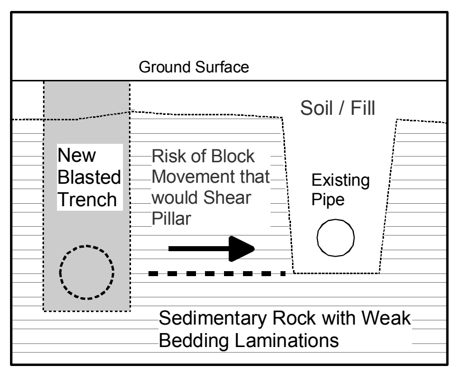
CASE 3 - GROUND-HEAVE FAILURE



Ground and geometry conditions indicate blast-induced ground heaving could damage the tunnel and pipes.

Consider:
1) Use multiple a series of smaller bench blasts in the adjacent excavation to reduce lateral pressure on the tunnel and pipes.
2) Apply minimum scaled distance limits to control charge size based on distance.
3) Use delay-decked charges to lift the ground and reduce lateral pressure.

CASE 4 - ROCK PILLAR FAILURE



Ground and geometry conditions indicate blast-induced ground heaving could shear the rock pillar and damage the pipe.

Consider:
1) Use open line-drilled or slot-drilled holes in the wall of the trench closest to the pipe.
2) Use delay-decked charges to lift the ground and reduce lateral pressure.
3) Installing vertical rock anchors to strengthen the rock pillar before blasting.

Figure 2. Ground and Geometry Cases with Higher Pipe Damage Risk

CASE HISTORIES

During the last 30 or so years, others have reported results from controlled tests and actual project work where blasting has occurred near pipes. Lewis L. Oriard, provides some very good case histories involving blasting near and even directly under pipes in "Explosives Engineering, Construction Vibrations and Geotechnology," (Oriard, 2002).

The following two case histories are offered to further demonstrate how blasting can be designed and done safely near pipes despite having PPV levels exceeding and sometimes far exceeding 125 mm/s (5 in/s).

CASE HISTORY 1: BLASTING NEAR A STEEL WATER MAIN

About 10 years ago, the author developed blasting plans for a Contractor that installed approximately 10,000 feet of steel pipes with diameters ranging from 18 to 21 inches. For much of the work, new pipes were installed in existing Right-of-Way easements. Very carefully controlled drilling and blasting methods were used to blast trenches in hard meta-volcanic rock located within 2.4m (8 ft) of an existing steel water pipe.

For this work, similar to demonstration Case 4 in Figure 2, the key challenge is to prevent rupture and movement of the pillar of rock in new and existing pipe trenches. It is also wise to assume that the 2.4m pillar of ground between the trenches has likely been fractured and somewhat over excavated when the existing pipe was installed.

With these concerns in mind, blasting was designed using very small delay-decked-charges initiated with 25-millisecond timing separation. Charge details are shown in Figure 3.

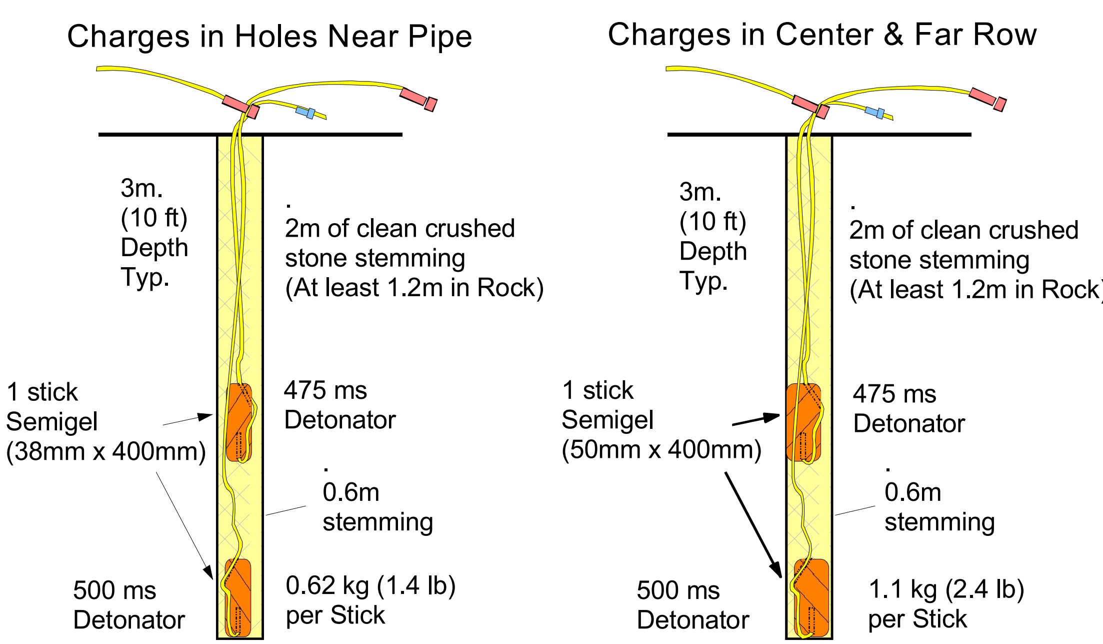


Figure 3. Arrangement of Delay-Decked Charges for Trench Blast near Water Pipe

As shown in Figure 4, holes were arranged in three rows. Center holes were offset and the spacing of holes in the row nearest to the existing pipe was reduced to from 1.4m to 0.91m to assure the smaller charges could break the hard rock. To minimize blast pressures against the rock pillar, the charge firing sequence was carefully designed so blasted rock would be lifted up and moved laterally down the trench and away from the existing pipe. The typical width of the blasts was 1m.

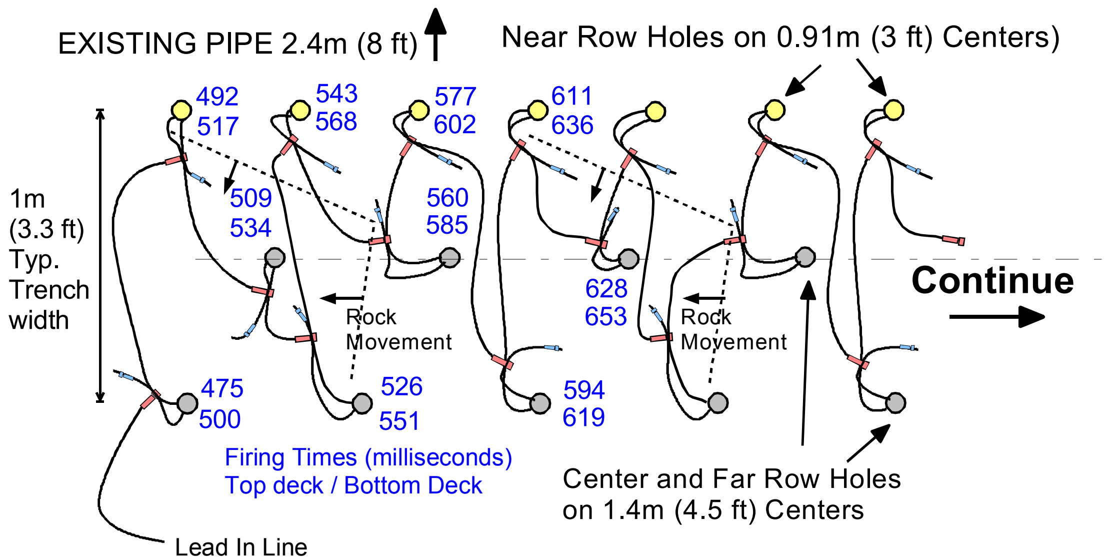


Figure 4. Drilling Layout and Delay Timing for Blast near Existing Water Pipe

When this work started, a 125mm/s (5.0 in/s) PPV limit was in place. Despite using very small charges with a maximum charge-per-delay of 1.1kg, PPV estimation calculations indicated PPV could exceed 125mm/s at 2.4m. Sure enough, in the eighth blast located 2.4m from the existing pipe, measured PPV on the existing water pipe was 203mm/s. The predominant frequency of motion was 30 Hz and maximum displacement was only 0.042 inches, so damage was very unlikely.

In response to the PPV measurement greater than 125 mm/s, the owner required the Contractor to suspend blasting so a camera inspection could be made in a 690-foot section of the steel pipe adjacent to the blast. The camera revealed the steel pipe was in good condition but an un-grouted joint from the time of initial installation was discovered. The Contractor was issued a change order and repaired it.

For subsequent blasting, motions measured on ground above the nearby pipe consistently had frequencies of motion ranging from 30 to 80 Hz. These high frequency motions indicated ground shifting was not occurring. After an onsite meeting to review all data and findings, the owner and construction manager later agreed to allow an increased PPV of 200 mm/s and the work would not be stopped until motion exceeded 250 mm/s. In hindsight, the original 125mm/s PPV limit was impractical for blasting work at a distance of 2.4m. All blasting work was completed and no pipes were damaged.

CASE HISTORY 2: BLASTING VERY NEAR STEEL PENSTOCK PIPES

In this case, tunnels and chambers were excavated in hard granitic rock to facilitate the installation of bypass pipes between two existing 2.43m steel penstock pipes and a new underground valve chamber. For minimal disruption to the hydropower plant, the final connections to each penstock were done during very tight shutdown periods scheduled a year apart.

Drill and blast methods were used to excavate access tunnels and chambers at the pipe connections. The Tie-in-Chambers were mined within 4m of the pipes while the penstocks were pressurized. The orientation of the pipe and a typical blast in the pillar of rock between the pipe and chamber is shown in Figure 5. Concrete backfill with an average thickness of 1.2m was placed in the annulus between the original excavation opening and the 14.3mm thick steel pipes when they were installed in the 1950's. Unlike normal pipes placed in trenches that are covered with relatively loose fill-material, these pipes were tightly secured to hard rock by the concrete.

When considering the very limited time to complete the work in rock with an unconfined compressive strength (UCS) of 125 MPa (18,125 psi), and the complex shape of the excavations, it quickly became apparent that the use of drill-blast methods was the only practical way to excavate rock in the access tunnels and connection chambers.

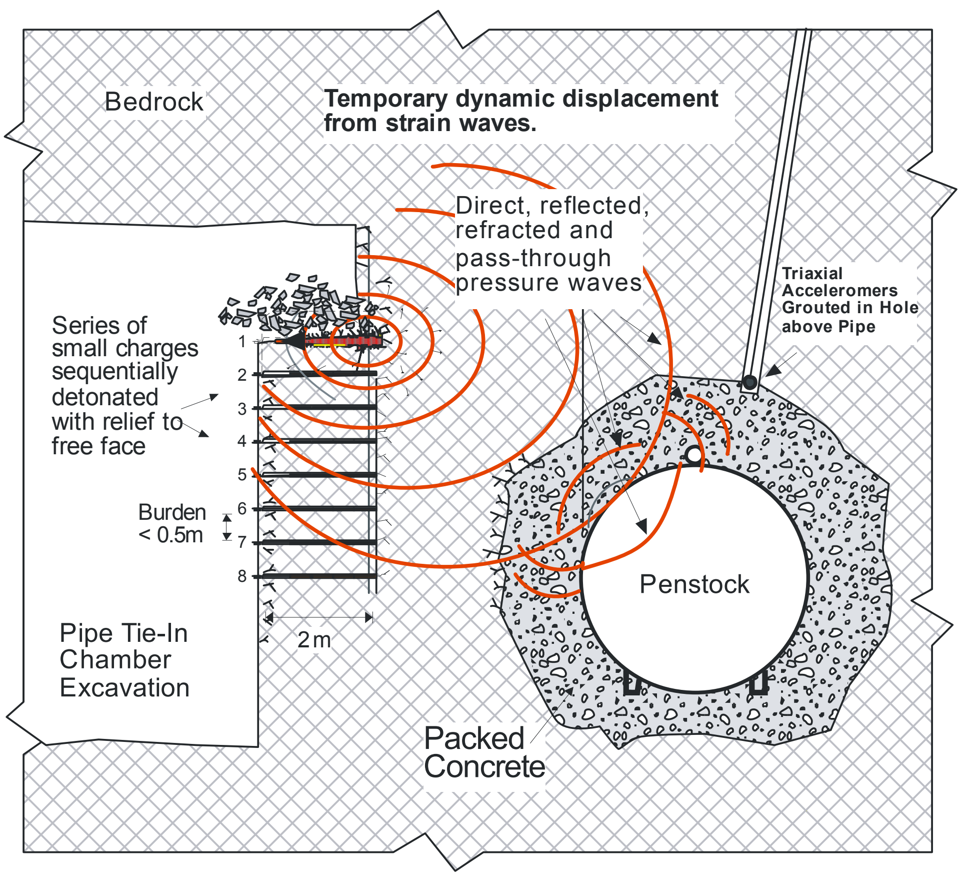


Figure 5. Configuration of Penstock and Blasted Connection Chamber

For this work, predicted particle velocities as high as 2,500 mm/s were expected at the pipes. Since this motion far exceeds the typical 125mm/s PPV limit applied to pipes, convincing engineers and managers involved in the work that blasting could be accomplished safely while working this far outside typical PPV limits was challenging.

In a scheduled shutdown before this work, a pipe was dewatered and an inspection revealed that the condition of the steel was very good. Early on, it was decided that the expected concrete with a thickness of 1m around pipes would be excavated by mechanical methods and blasting done near the pipes must be designed to assure blast-induced pressure would not cause direct rupturing of the penstocks.

Very rigorous specifications were developed to control the blasting in various zones of the excavation work. These controls included limitations of charge configurations, minimum scaled distance requirements, and a maximum dynamic displacement limit of 1mm at the pipe. The displacement limit was based on modeling of microstrain (μmm/mm) due to pipe bending.

Since vibration monitoring equipment cannot directly measure displacement, and velocity transducers could not handle the expected high particle velocities and frequencies of motion, accelerometers with a 500g (4.9 x 10^6 mm/s^2) and 1Hz to 3kHz range were used to perform primary measurements of ground motion. As shown in Figure 5, triaxial accelerometers were grouted in drilled holes located above the pipes.

Displacements were estimated using sinusoidal relationships. Since all blasts would have varying proximity to the closest section of pipe and the measurement points, extrapolation calculations were used to predict the intensity of motions at the closest part of the pipe. For extra caution, only upper 95% confidence curves derived from site measurements were used to estimate blast effects at the pipes.

To provide additional data for site scaling curves, special triaxial velocity transducers with a 1kHz frequency and 2,540 mm/s range were used to measure particle velocities at various locations in the chambers.

For the very close-in blasting that occurred 3m to 1m from the pipe, there were rigid controls on charge configurations and charge relief. To prevent block motion in the 1m or more of rock/concrete between blasts and the pipes, burden was limited to 0.5m and extremely light charges assembled from detonating cord and a small primer cartridge were used to reduce borehole pressures.

As expected, due to the extremely confined condition of the steel pipes confined by concrete in rock, the intensity of particle acceleration measured in ground near the pipes was very high. As shown in Figure 6, accelerations were as high as 478 g's (4.69 x 10^6 mm/s^2).

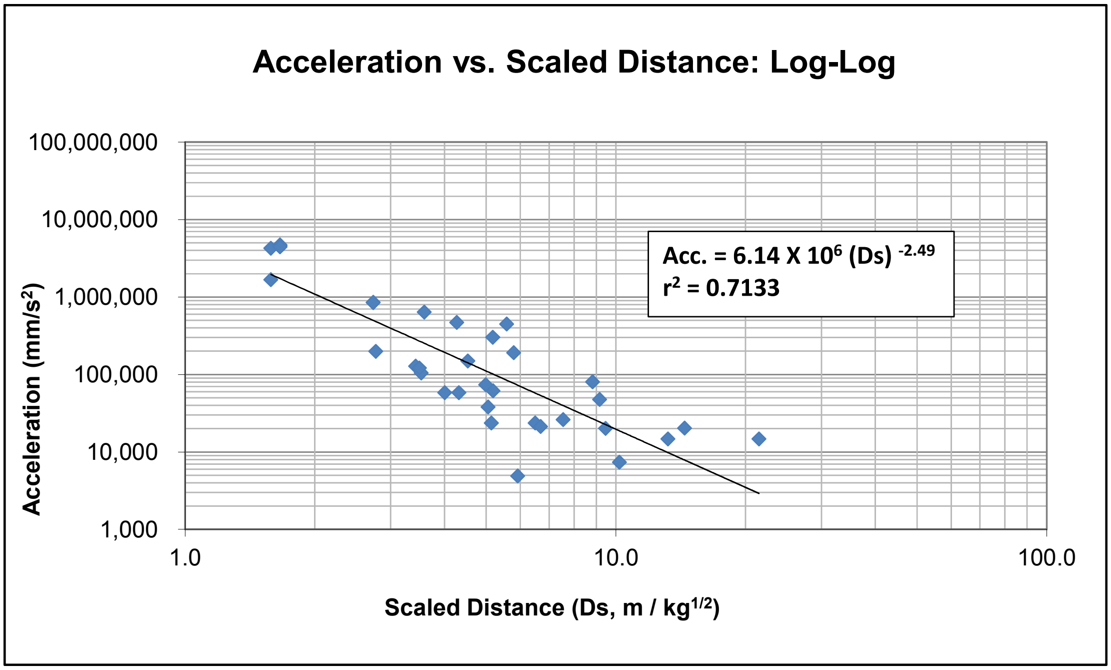


Figure 6. Acceleration Data and Best-Fit Curve

Three measurements at 1.4m also reached the 3kHz limit of the accelerometers. However, as shown in in Figure 7, the three data points, clustered in a group at 1.4m, are well above the trend curve. Hence, these 3kHz frequency measurements may be anomalous due to the sensor approaching its 500g-acceleration limit.

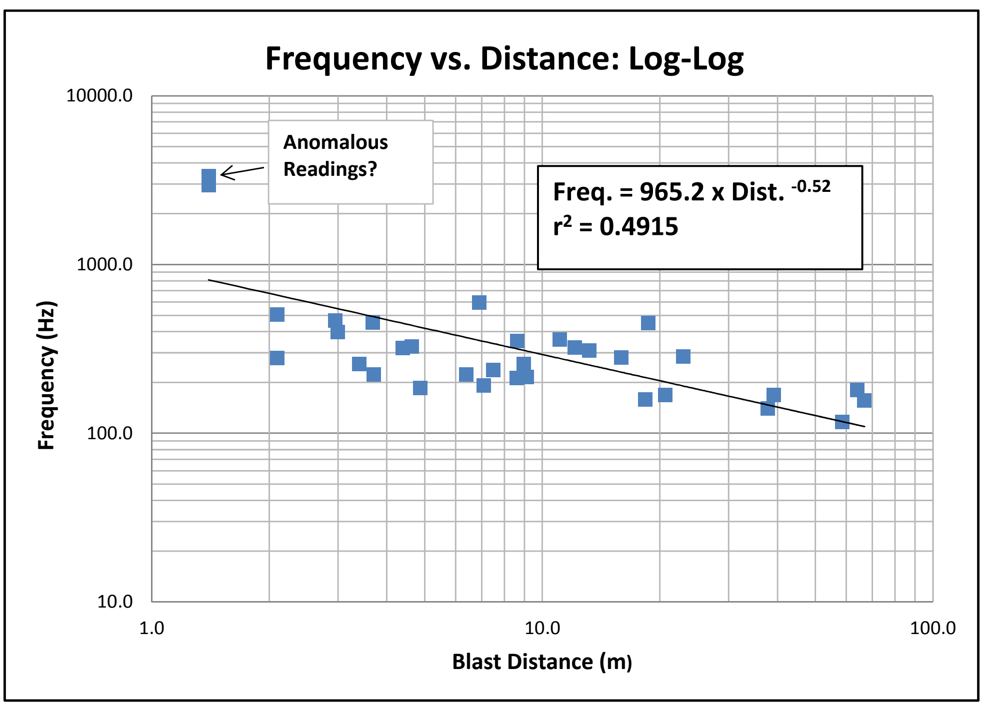


Figure 7. Frequency of Motion Data and Best-Fit Curve

Results. In the first summer of work, blasting and mechanical excavations for the first penstock connection were successful. In the second summer, when concrete around the pipe was removed mechanically, a smooth dent was found in the pipe. Investigations of the area revealed a portion of remnant open hole near the damaged area. This indicated a blasthole was improperly drilled too close to the pipe and the dent was caused by direct gas pressure created by the charge. The dent was repaired when the cutout was made for the connection.

In a few instances, the estimated displacement at the pipe did exceed 1mm, but never exceeded 1.5mm. Since this displacement limitation had a reasonably high factor of safety there was no concern about damage to the pipe.

It was very evident to everyone involved in the work that the primary concepts applied to protect the pipe from blast-induced damage were effective. The one incident where the pipe was dented in one of the last blasts highlights the importance of carefully overseeing all work to assure complete compliance with all critical limitations and controls.

CLOSING REMARK

From this review, it is hoped that readers will remember this: High PPV motion occurring at high frequencies rarely damages pipes or structures; more importantly, it is an indication that potentially damaging heaving motion has not occurred!

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Completion and Startup of Utah Lake System Pipelines

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Abstract

Over the last decade, the Central Utah Water Conservancy District (CUWCD) has designed and constructed 13 projects to complete 32 miles of large-diameter pipe and flow control structures for the Utah Lake Drainage Basin Water Delivery System (Utah Lake System), which represents over $200 million of constructed facilities. The Utah Lake System is a component of the Bonneville Unit of the federal Central Utah Project, which delivers municipal water to major cities in northern and central Utah. Water is collected within the Colorado River Basin on the east side of the State and conveyed via reservoirs, tunnels, and pipelines to the Great Basin along the Wasatch Front. The Bonneville Unit is comprised of three major projects:

1. The 40-mile long Strawberry Aqueduct Collection System which collects water from numerous drainages extending from Stillwater Reservoir on Rock Creek to Strawberry Reservoir through a system of tunnels and pipelines;

2. The 19-mile long Diamond Fork System which is a transbasin diversion that conveys water collected and stored in Strawberry Reservoir within the Colorado River Drainage to the Great Basin via tunnels and pipelines up to 660 cfs capacity; and

3. The Utah Lake Drainage Basin Water Delivery System which conveys and distributes water conveyed from Strawberry Reservoir to municipal water users along the Wasatch Front.

In addition to municipal water deliveries, water is delivered to some of the streams and rivers within the Great Basin to provide supplemental flows for fish and wildlife. In the spring of 2015, two of the three branches of the Utah Lake System will be substantially completed and brought into service, which will result in a 124-mile gravity flow system from Stillwater Reservoir via Strawberry Reservoir to water users in Salt Lake City. This paper presents the to-date completed Diamond Fork and Utah Lake System aqueducts and the project's unique challenges related to planning, design, and construction. Two of the three major pipeline branches of the Utah Lake System are constructed at this time. The completed portions of the Utah Lake System includes 26 miles of 96-inch and 60-inch–diameter welded-steel pipe at 280 to 450 psi pressures and 6 miles 54-inch HDPE pipe. The project includes six major flow control

structures designed to deliver up to 120 cfs and break pressures of 420 psi to atmospheric conditions. Other major facilities include pig launching and receiving facilities for the 96-inch and 60-inch pipelines, a 6.7-MG rectangular concrete regulating reservoir, and 15 separate turnout vaults designed to deliver water to existing flood irrigation and municipal secondary systems with the option to convert to pressurized delivery in the future. Topics discussed include crossing and construction near the Wasatch fault, landslides, cavitation control at high-head turnouts, flow and pressure control between the upper and lower delivery systems, and maintaining system capacity with pigging facilities.

UTAH LAKE SYSTEM OVERVIEW

Location and Function. **Figure 1** (next page) shows where the Utah Lake System (ULS) is located in northern Utah and shows that this system connects together several major upstream (Diamond Fork) and downstream (Salt Lake, Jordan and Provo River Aqueducts) aqueduct systems. The ULS also joins the largest fresh water storage reservoir in northern Utah (1.1 million acre feet Strawberry Reservoir) to CUWCD's customers in municipal and rural areas of both Utah and Salt Lake counties.

Several Flow Control Stations (FCS's) control the system. The larger of these stations are shown in plan and profile in **Figures 1 and 2**.

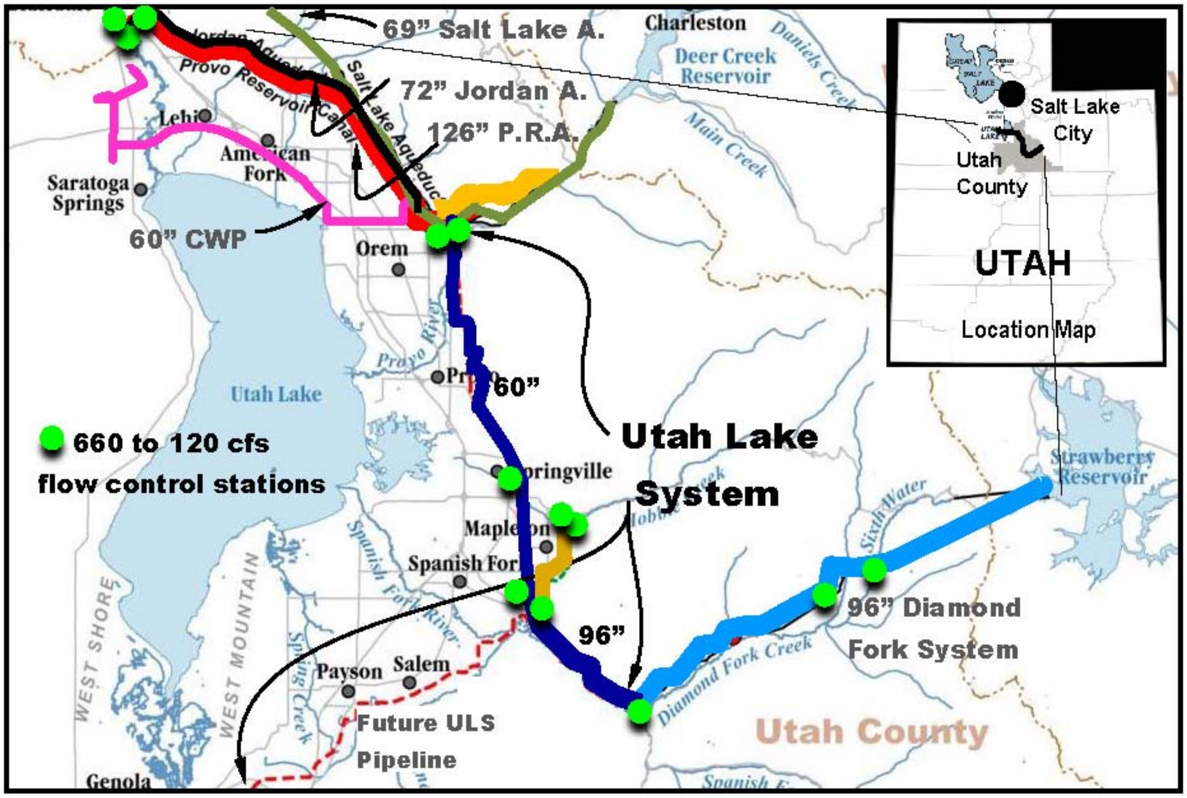
The Utah Lake System is comprised of a 96-inch, 365 cfs capacity Spanish Fork Canyon Pipeline that conveys water to the populated valley areas of the Wasatch Front. At the mouth of the Spanish Fork Canyon, the pipeline branches into three pipeline segments as described below:

- The Spanish Fork Provo Reservoir Canal Pipeline is a 20 mile long, 60-inch welded steel pipeline wth a capacity of 120 cfs that delivers water to the Provo River and to aqueducts near the mouth of the Provo River Canyon that convey water to treatment plants in Salt County where water is distributed to municipal users. (Completed)

- The Mapleton Springville Pipeline is a 6 mile long, 54-inch HDPE pipeline with a capacity of 125 cfs that replaces a canal constructed in 1918 that delivers water to multiple laterals for irrigation and secondary water uses and delivers water to Hobble Creek to restore spawning habitat for an endangered fish. (Completed)

- The Spanish Fork Santaquin Pipeline is a 17 mile long, 60-inch to 36-inch welded steel pipeline that delivers water to municipal secondary systems. (Not Yet Constructed)

FIGURE 1
The Utah Lake System connects the four largest Utah aqueducts to 1.1 million AF of storage.

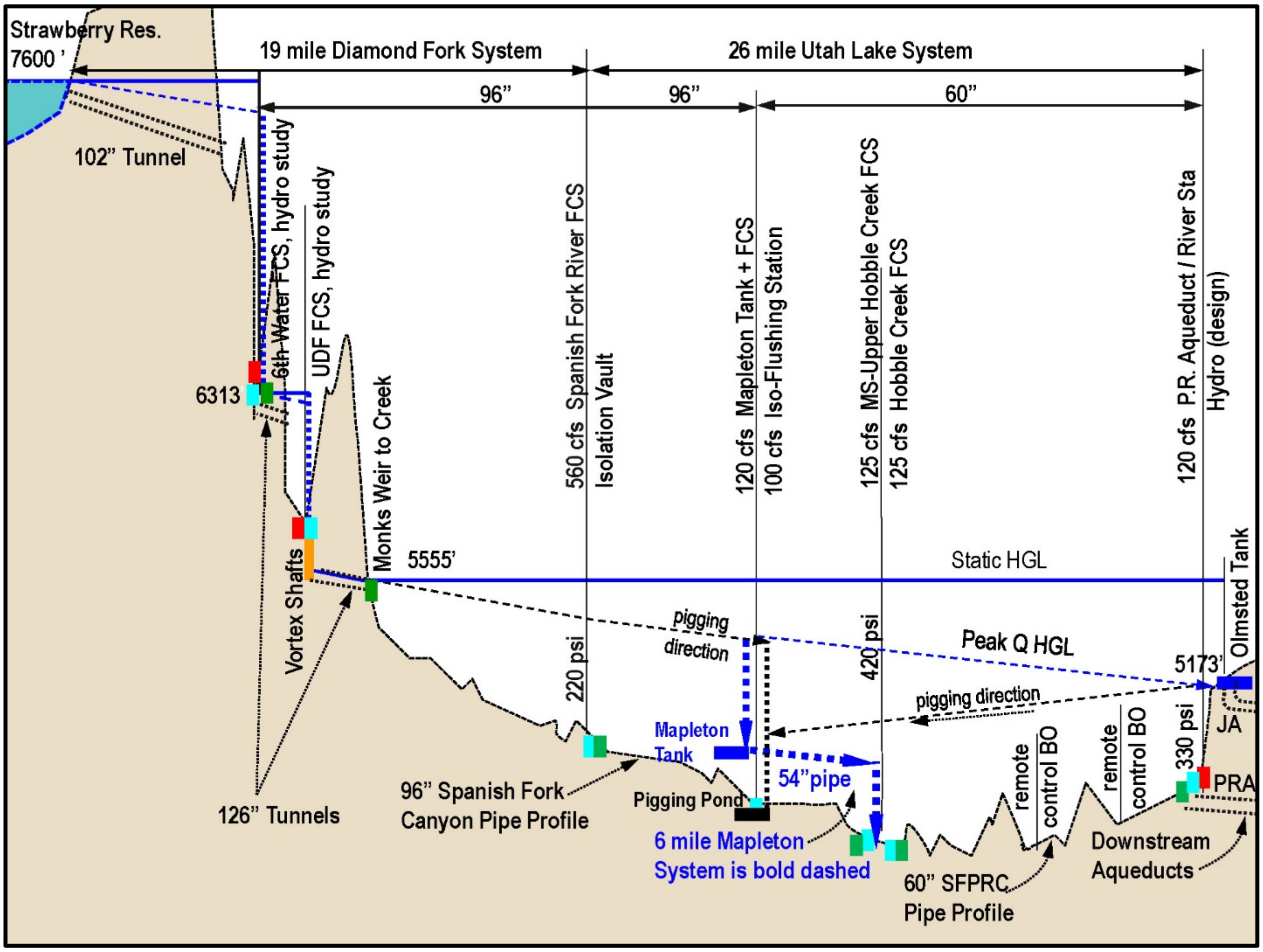


System Hydraulic Profile. The hydraulic profile for the northern branch of the combined Diamond Fork-Utah Lake System is shown in **Figure 2** (next page). Also shown on Figure 2 in the lower center is the hydraulic profile of the Mapleton Springville Pipeline. Two 660 cfs flow control stations (FCS's) in the Diamond Fork System break the Strawberry Reservoir hydraulic gradient (elev. 7600 ft) down to the main ULS pressure zone, which is designed with an upstream weir at Monks Hollow (elev. 5555 ft).

The Monks Hollow weir, maintains the system gradient by a constant "trickle" flow over the weir. This trickle flow over the weir allows the Diamond Fork-Utah Lake System to operate at system full (steady pressure head) without interim storage while delivering water over the weir to maintain minimum instream flows.

High and Low Head Flow Control Stations and Cavitation Control. The main ULS pressure zone (HGL 5555 ft) has several high head flow control stations (FCS's). Some discharge "to atmosphere" into an open basin. Two discharge "in-line" into a pressure pipeline. The lower Mapleton System pressure zone (HGL 4985 ft) also has fifteen (15) automated flow control vaults, varying in size from 125 cfs to 7 cfs. These are low head stations, many of which discharge both to atmosphere and into pressure irrigation systems. Station layouts and control valves are fitted to head, flow, and cavitation control needs special to each station.

FIGURE 2
The Diamond Fork and Utah Lake Systems (hydraulic profile below) act hydraulically as one system. To date, the Utah Lake System includes three pipelines shown in the profile: the 7-mile 96-inch Spanish Fork Canyon Pipeline, the 60-inch 19-mile SFPRC Pipelines, and the 54-inch 6-mile Mapleton-Springville Pipeline (or "Mapleton Pipeline"). The first two form the main ULS pressure zone (HGL 5555 ft) which has 3 flow control stations (FCS) and a pigging station, which each deliver 120+ cfs. The Mapleton (HGL 4985 ft) is shown bolded in the lower middle.



High head break stations with vertical sleeve valves discharging into basins include:

- Mapleton Tank FCS breaks 250 psi for 120+ cfs into Mapleton Tank with 2 sleeve valves
- Hobble Creek FCS breaks 420 psi for 120 cfs to Hobble Creek with 1 sleeve valve.

FIGURE 3. *Hobble Creek Station valve room. The sleeve valve is below floor at the back of the photo.*

High head break stations with plunger valves discharging to low head downstream pipes include:

- Orem 1B FCS breaks 30 to 300 psi for 120+ cfs into the 126" PRA pipe with 4 plunger valves (**Figure 7**). This station is also the pig launch site for the 60-inch ULS pipe cleaning.
- Orem 1B FCS also breaks 5 to 330 psi for up to 120 cfs discharge to Provo River. A future tie to Jordan-Olmsted system will allow the plunger valve to deliver 120 cfs to that system.
- IsoFlushing FCS (**Fig. 5**) breaks 320 psi at 100 cfs to flush ULS pipe sludge water to a pond.

Low head (80 psi) stations in the Mapleton system include 12 stations with 10-inch to 16-inch globe valves which break head either into open ditches (valves have cavitation trim and downstream orifice plates) or into pressure pipelines. Some vaults have two parallel control lines to allow both functions at the same station.

The 400 North, Springville and Upper Hobble Creek (see **Figure 4**) Stations, deliver 35, 70, 125 cfs respectively, to downstream pipes with low-head (6 to 8 psi). Each has plunger valves to handle high flows and low back pressures. The two Springville Station valves, originally in the Upper Hobble Creek Station, especially had adverse downstream cavitation which would have required 42-inch plunger valves, so to reduce valve costs, the two valves were moved downstream to a separate vault to increase backpressures and downsize valves to 28-inches.

FIGURE 4. *Above right is Upper Hobble Creek Station valve room being built with 36- inch process lines. The plunger valve discharges up to 120 cfs discharge to a low head basin.*

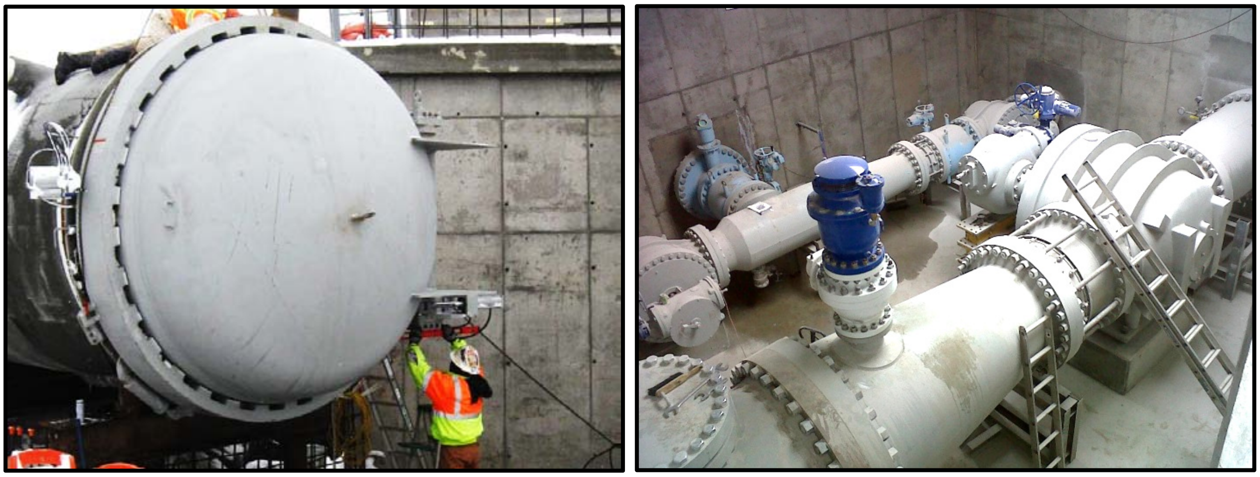
PIGGING RESTORES ORIGINAL SYSTEM CAPACITY

Over the last 60 years, Utah's Aqueduct operators have learned that, over time, most Wasatch Mountain surface waters generate a dark brown organic slime on pipe linings. Over a 20 year period this slime thickens and has caused a 15 to 30 percent capacity loss in the Salt Lake, Jordan and Olmsted aqueducts. Decades of testing ways to eliminate these losses has found that mechanical pigging is the best way to remove the slime and restore original pipe capacity.

Although design-flows drop-off at the downstream end of the ULS and smaller pipe diameters might have been used, to allow pigging, the system was designed with only two pipe diameters. From Monks Weir to the IsoFlushing (pigging) Station was sized for a 96-inch pig. A "dumbbell pig" is planned because a 96-inch foam pig weights 25 tons when wet. From the IsoFlushing Station to the downstream aqueducts connection, the pipe was sized for a 60-inch foam pig.

A 100 acre-foot pigging pond was provided at the IsoFlushing Station so turbid pigging water from both the upper 96-inch and lower 60-inch pipelines could be discharged and infiltrated, leaving the native surface water organic slime to dry out and be used as an organic fertilizer. Note the 60-inch pipeline is pigged in reverse direction to normal flows, by using Olmsted Aqueduct water diverted from the Provo River at a higher hydraulic gradient than IsoFlushing Station location. **Figure 5** shows IsoFlushing Station facilities.

FIGURE 5
IsoFlushing Station includes 108-inch and 66-inch pig retrieval doors (one at lower right), a valve room (lower left) with a 42-inch cone valve and two 24-inch, 330-psi plunger valves (at back of valve room) to flush "slime" water from both pipelines into pigging pond.



SYSTEM RELIABILITY AND RISK

The ULS design provided for system reliability and risk reduction are critical areas, including addressing city concerns for pipe break risks and locating critical storage and flow control stations away from faults and on hard bedrock.

Addressing Risk of Pipe Breaks in Cities. The ULS high pressures (330 to 450 psi) in city streets generated comments of concern from the cities through with the pipelines passed. The risk of pipe break is extremely low due to it being well protected by features such as the top of pipe being typically 11 to 18 feet deep (below sewers), and the double welded steel pipe is thick (0.6 to 0.7-inches) with double coatings (mortar and dielecteric) with cathodic protection. However to further reduce risk, CUWCD prepared an emergency action plan and added remote controlled 16-inch blowoffs at large storm drains to supplement the 120-cfs draining capacity provided at the Hobble Creek Station. These are located in the hydraulic profile (see **Figure 2**) and in **Figure 6**.

FIGURE 6.
Above right is an inside view of one of the 16-inch automated blow-off valve vaults.

Addressing Fault Risks at Mapleton Springville Tank. As part of the ULS, the 6-mile long Mapleton -Springville Canal was piped. A 6.7 MG regulating tank (see **Figure 7**) was added in between the 320 psi ULS pipeline and the 80 psi Mapleton Irrigation System. A hill by the juncture of these two systems was investigated for the tank site and found to include active faults. However, through literature study, and field geologic explorations (several trenches and holes), the fault locations were well documented and the tank site was located with minimum risk on the uphill side of the located faults.

FIGURE 7
The 6.7 MG Mapleton Tank was sited to remain over 50 feet from a splay of an active fault. The active fault runs beneath the row of cars on the right of the tank. This photo was taken just before leak testing and burying the tank. The ULS field office is in the back upper right.

Several provisions were made to increase the system reliability, and to reduce risks of both the tank and the inlet-outlet piping. These included:

- the tank was designed with an underdrain system with 23 “cells” to monitored drain pipes
- foundation soils which might consolidate were removed and replaced with concrete fill
- a high head shutoff valve was built on the 320 psi inlet pipe upstream of the fault
- the flow control station breaking head to the tank was built monolithic with the tank
- a 42-inch HDPE pipe was built to drain the tank to the pigging pond in an emergency
- remote controlled isolation valves to the 54-inch Mapleton Pipe and 42-inch drain were provided in vaults built monolithically with the tank

Addressing Fault Risks at Orem 1B Station. The Orem 1B flow control station connects the 60-inch ULS pipe to the 126-inch Provo River Aqueduct (PRA) where it exits Provo Canyon and crosses the Provo River. The first plan was to site the station on a hill 150 feet above the river to allow for an atmospheric discharge to a basin which would then flow into the PRA.

When the hill geology was investigated and test trenches dug, three fault splays were found crossing the desired hilltop station site. The only corner of the site not covered with active faults on the limestone uphill of the fault was beneath high voltage power lines. All the other sites put the station over fault splays.

For these reasons, the Orem 1B Station was moved down the hill onto the hard limestone bedrock east of the fault and beside the Provo River and PRA connection. This added more costs for valves but reduced land costs and offsite access costs. More importantly, it put the station on hard rock foundation which would minimize station damage during a fault movement. The Orem 1B Station is shown at the base of the hill in **Figure 8.**

FIGURE 8
The Orem 1B Station is the terminal connection station which delivers flows from the 60-inch ULS pipe into the 126-inch PRA pipeline. The station is right beside the Wasatch Fault but is founded on hard bedrock to protect it from significant damage in a fault movement event.

Because the upstream ULS pipe crosses the fault and Provo River concurrently, in a fault rupture, the pipe can discharge its full capacity to the river with little risk because the river capacity exceeds that of the pipeline by more than 50 times. This Orem 1B Station site also aligned the pipeline connection to the Olmsted-Alpine-Jordan Aqueduct System onto hard bedrock.

Addressing Landslide Risks by the SFFCS Station. Due to the need to cross a landslide prone formation at the top of the Utah Lake System, an isolation valve vault **(Figure 9)** was built to allow the system to deliver water through the Spanish Fork River Flow Control Station if the ULS pipeline were to be ruptured due to a landslide.

FIGURE 9
The SFFCS valve station at right is shown under construction. This station connects the Diamond Fork and Utah Lake Systems and allows passage of a 96-inch pig through the station.

SYSTEM SIMPLICITY AND FLEXIBLITY

Although the Diamond Fork-Utah Lake System has no storage in its main 5555 pressure zone, storage is provided below that zone in both the Mapleton System and downstream Jordan-Alpine-Olmsted System.

The 6.7 MG Mapleton Tank (see **Figure 7**), is located at the head of the 125 cfs Mapleton System and allows this sub-system to change flows independently without reflecting those flow changes upstream into the 5555' and 6310' pressure zones.

Operating storage will also be available in the Jordan-Alpine-Olmsted System where the existing 10 MG Olmsted Tank is located (**Figure 10**) just downstream of the Utah Lake System. The Olmsted System is the water source for pigging the 60-inch ULS pipeline.

FIGURE 10
At upper right, the 10 MG Olmsted Tank is shown before backfilling. It was constructed as part of the Olmsted System 12 years ago. Because it is at the ULS downstream connection, it is able to provide operating storage for the Utah Lake System also.

Valve Closure Times and Surge Control. To keep surge pressures low in the 35- mile long 5555’ pressure zone, all its valves are designed to open and close with half-hour or greater closure times.

UTAH LAKE SYSTEM STARTUP

In June of 2015, the Utah Lake System, which has been under construction since 2007, will begin deliveries through the following new system components:

- the fully functional, pressurized Mapleton System
- the 60-inch SFPRC Pipeline. This pipeline is the downstream end of the 5555’ pressure zone, and now includes connections to the 126-inch Provo River Aqueduct and to Provo River. This connects CUWCD’s contracted customers in Salt Lake County to the Strawberry Reservoir storage.

Additional functions, such as connecting the ULS to allow deliveries into the higher gradient Jordan-Alpine-Olmsted Aqueduct System, and conveying water to secondary systems of municipal water users in south Utah County are anticipated in the next few years.

Compacting Pipeline Embedment Soils with Saturation and Vibration

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Abstract

Proper buried pipeline installation relies on support for the pipe in the haunch area. This is necessary for all types of pipe material. One method of obtaining haunch support is by using a cohesionless soils and providing compaction by using saturation and internal vibration (jetting and vibrating). The method uses internal concrete vibrators and enough water to lubricate the soil particles. The method has two significant advantages: (1) the soil in the pipe haunch area can be effectively compacted to a high density, and (2) the compacted lift thickness can be several feet thick, limited by only the length of the vibrator. Denver Water has been utilizing this method since the 1960s. US Bureau of Reclamation has been using the method since the 1950s and at one time saturation and vibration was their only acceptable compaction method for pipe embedment. This paper will describe research investigations and present case studies on the use of compaction of cohesionless soils by saturation and internal vibration.

INTRODUCTION

Well compacted cohesionless soils provide the best support for buried rigid and flexible pipe. Clean sands, gravels, and crushed rock are typical cohesionless soils used in pipeline construction. These soils contain few fines and are best compacted using vibration. Vibration shakes the soil particles, shifting them into a denser arrangement. Vibration works best for clean sands and gravels (containing less than 5% fines). Vibratory compaction equipment includes *vibratory drum rollers, vibrating surface plates*, and *insertion (or internal) vibrators.* Vibratory drum rollers are steel smooth drum, sheepsfoot, or padfoot rollers that have vibrating drums. The pressure or kneading action is combined with vibration. Small vibrating drum rollers, either *walk-behind* or *ride-on* models, are used for trench compaction. They have either a smooth drum or a dimpled drum for traction. This paper describes the saturation and vibration (terminology used by US Bureau of Reclamation) and jetting and vibrating (terminology used by the Denver Water Board) method.

PROCEDURE

Clean sands and gravels can be compacted in lifts of 6 feet *(2 m)* or more using the *saturation and vibration* technique. In this method, water is added to the soil and internal vibrators, such as the concrete vibrator shown in Figure 1, are worked down through the depth of soil that was placed.

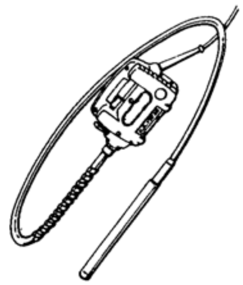

Figure 1 Internal Vibrator

Internal vibrators are also known as stingers, concrete vibrators, or wiggletails. The depth of effective compaction is only limited to the length of the vibrator. A pneumatic vibrator used by a contractor on a Denver Water Board project is shown as Figure 2. Note the solid vibrating head and the flexible shaft.

Figure 2 Internal Vibrator Used on Denver Water Board Project

Compaction using internal vibrators in cohesionless soils is described in ASTM F 1668 *Practice for Installing Thermoplastic Pipe*. A video of the procedure can be seen on the links page of the website Pipeline-Installation.com.

A photo of a typical saturation and vibration operation is shown as Figure 3.

Figure 3 Saturation and Vibration Operation

The effectiveness of the internal vibrator depends on:

- The size of the vibrator head.
- The frequency and amplitude of vibration.
- Whether vibrator is pneumatic or electric (pneumatic is more effective).
- What point in the operation the water is added to the soil.
- The amount of water.
- The spacing between vibrator insertions.
- The speed at which the vibrator is withdrawn.

All of these factors, plus the specific type of soil that is to be densified, require that time be spent at the beginning of the job (and when any of the factors change) experimenting to find the best combination that will compact effectively and efficiently. However, being able to compact very thick lifts means that this trial period can be extremely worthwhile. Unfortunately, for contractors unfamiliar with this method, the experimental period can be unsettling and may take more time than anticipated. While the method has been called *saturation* and vibration, only enough water should be added to lubricate the particles for ease of densification. If too much water is used when compacting the soil on the sides of a pipe, it is possible to float the pipe. The saturation and vibration method is very effective in densifying soil in the haunch area, even if the tip of the vibrator is not close to the haunch.

This method may be more effective if the soil is added to water, rather than vice versa. In this way, the water is worked up through the soil, not down through the soil. It is harder to work air out of soil when water is over the air.

PIPE HAUNCH AREA

The haunch area of the pipe is particularly difficult to compact soil into. A recent study showed how just dumping the soil in can create a void in the lower portion of the haunch (Boschert and Howard 2014). This is illustrated in a video at Pipeline-Installation.com and go to >links >videos from pipe seminar >Creating lack of support in pipe haunch.

The best solution for ensuring good haunch support is to use flowable fill (even if it is not used for the rest of the pipe embedment and backfill). The next best method is to use cohesionless soils compacted by saturation and vibration. Reclamation field tests have demonstrated that high densities could achieved even thought the tip of the vibrator was several feet away from the haunch.

APPLICABLE SOILS

Table 1 shows the Uniform Soil Groups that are used for pipeline installation (Howard 2009). Cohesionless soils are the soils in Class I and Class II - clean sands, gravels, and crushed rock.

Class I	crushed rock	"GP"
Class II	clean, coarse grained soils (includes dual symbol soils starting with one of these symbols, e.g., GP-GM)	GW, GP, SW, SP
Class III	coarse grained soils with fines sandy or gravelly fine grained soils	GM, GC, SM, SC sML, sCL gML, gCL
Class IV	fine-grained soils	ML, CL
Class V	fine-grained soils, organic soils	MH, CH, OL, OH, Pt

Table 1 Uniform Soil Groups for Pipeline Installation

Some sands in the dual symbol category (5% to 12% fines).would be best compacted by impact, pressure, or kneading. Trial and error may be required to determine the best compaction method. Determining the best method of compaction (results in highest density) is explained in Howard (2015).

The McGee Creek case study described later in Case Histories describes how important the percentages of fines are for successful compaction with vibration.

"SELF COMPACTING" SOILS

Gravels and crushed rock are sometimes referred to as *self-compacting* meaning that if they are dumped in beside a pipe from enough height the soil will have a high density. However, these soils actually have a density close to their minimum density which is about 80 to 85% of their maximum density. The amount of soil support for buried pipe is directly related to the stiffness of the soil. Increasing the density of gravels from 85% of their maximum density to 95% can easily double their stiffness.

Raining cohesionless soils (pluviation) in laboratory testing is sometimes used to create high densities. In the laboratory, individual soil particles fall vertically without interference onto other particles resting on the surface and the resulting impact results in compaction. This is not the case in the field. Particles that are dumped into place roll, slide, and collide reducing the compaction. Furthermore, in a pipe trench, dumped gravel impacts the pipe and the trench walls in addition to particles hitting each other. Consequently, the density is substantially reduced.

In all of the pipe installation manuals/standards published by AWWA, ASTM, and ASCE, there is no mention of dumping soils to get a high density. Neither is there any such mention in any of the pipe installation guides published by the pipe trade associations. There is no technical data published that demonstrates that the gravel is densified by dumping. However, there are numerous claims in internet discussion boards by contractors that gravels are *self-compacting*, but engineer contributors disagree.

Clean, cohesionless soils such as gravel and sand are sometimes evaluated, or specified, by Relative Density. In the Relative Density (RD) procedure, laboratory tests are used to determine the minimum and maximum density of a clean sand/gravel. The field density is compared to both densities and calculated to be somewhere in between, such as "62% RD." For pipeline support, the value 70% RD is considered to be roughly the equivalent of 95% standard Proctor. Several sources have compiled data on the minimum density of a soil and the maximum density of the same soil, typically gathered from the Relative Density test (ASTM D 4253 and ASTM D 4254). The minimum density should be close to the density of a soil that is dumped into place. The data shows that the minimum density is generally 75% to 85% of the maximum density with 80% being a representative average. These studies are detailed in the Technical Note "Self-Compacting Soils – Not !" on the download page on the website Pipeline-Installation.com.

These comparisons mean that gravels and crushed rock that are dumped into a pipe trench are typically only about 80% of their maximum density. Although 80% sounds like a significant number, the dumped gravel and crushed rock are, in fact, being placed close to the lowest possible density and consequently placed at a minimum stiffness.

One study found that: % compaction = 80+0.2 RD
[or RD = 5 (% compaction/100) – 4]

where RD is the Relative Density in percent (Lee and Singh 1971). So an RD of 60% would be equivalent to 80+0.2(60) – or 92% compaction. This serves as a handy rule of thumb for comparing percent compaction and relative density for cohesionless soils. The other implication of this guideline is that dumped soil (0% RD) is 80% compaction.

HISTORY

Bureau of Reclamation

In 1955 and 1956, field and laboratory tests were conducted on the feasibility of using internal vibrators to compact cohesionless pipe embedment soils (Sailer 1955) (Holtz 1957). The results of the field study showed that compared to the traditional cohesive soil compacted in 6 inch thick layers, the vibrated cohesionless soil:

- Required less space between pipe and trench (reduced trench width).
- Reduced trench width meant less excavation and less backfilling.
- Was easier to place around the pipe.
- Provided better pipe support with less deformation.
- Provided more uniform support for the pipe.
- Was more economical to place (one lift rather than many).

Laboratory compression tests on various soils and various densities showed:

- 70% Relative density was equivalent to 95% (ASTM D 698).
- Stiffness of cohesionless soils below 70% decreased significantly.
- Stiffness of cohesionless soils above 70% increased moderately.
- Vibration worked best if fines content was 8% or less.
- Some soils up to about 12% fines could be vibrated successfully.
- Surcharge weight increased effectiveness of vibration (greater depth gave higher densities)
- Adding water was essential in obtaining high densities in cohesionless soils by vibration.
- Stiffness of cohesionless soils increased with increasing gravel content.

The method was referred to as "saturation and vibration" and was listed as an option in Reclamation specifications for compacting cohesionless soils for pipe embedment.

Denver Water Board

In 1960 Denver Water began to use clean sand around pipe in the pipe zone for pipe with a diameter larger than 24 inches. This embedment was compacted by jetting and vibrating. Denver Water refers to the method as *jetting* because the contractors typically use a separate rigid pipe that is placed down into the sand adding water to facilitate the compaction as the particles of sand were moved around by the vibration. The procedure is illustrated in Figure 4. The inserted vibrator is shown on the left side and the jetting pipe on the right side of the photograph.

Figure 4 Using Jetting Insert and Vibration on Denver Water Board Project

A concrete vibrator was used as illustrated earlier in Figure 2. The rigid pipe insert is shown in Figure 5.

Figure 5 Rigid Insertion Tool for Jetting Water for Denver Water Board Project

Currently, the lift thickness is limited to 4 foot if jetting and vibration is used.

No gradation was specified for the sand at the time. Experience with the process and attainment of the specified density dictated the gradation. Only enough water was introduced to attain the specified density which varied from 60-85% Relative Density (ASTM D 4253 and D 4254) depending of the location of the pipeline. The sand came from gravel pits along the South Platte River. The quality of the material varied depending on how far the pit was from the mouth of the South Platte River Canyon near Waterton, CO. The material mined near the mouth contained a lot of fines and was difficult to consolidate. Material mined further downstream was cleaner and produced the desired results. This resulted in the limit of 3% fines (minus No. 200) material in the subsequent allowable gradation.

In 1976 Denver Water started listing the sand gradation in project specifications. This was done to obtain consistency in the sand supplied so that the consolidation could be accomplished with a minimum of arguments with the contractors. The sand gradation was listed as follows:

Sieve Size	Total Passing by Size (Percent by Weight)
3/ 8 inch	100
No. 4	70-100
No. 8	36-93
No. 16	20-82
No. 30	8-65
No. 50	2-30
No. 100	1-10
No. 200	0-3

Currently the Denver Water Engineering Standards list the requirements for pipe zone material as being clean, free draining, well graded sand with the following gradation:

Sieve Size	Total Percent Passing by Weight
3/8 inch	100
No. 4	70-100
No. 8	36-93
No. 16	20-80
No. 30	8-65
No. 50	2-30
No. 100	1-10
No. 200	0-3

The Denver Water Board has installed about 2500 miles of pipe; 500 miles of pipe larger than 24 inches in diameter. About half of that has been installed since 1960 using the jetting and vibrating method. The largest pipe installed using this method is 144 inches.

CASE STUDIES

Bureau of Reclamation - Gradation

About 1980, Reclamation only allowed clean gravels compacted by saturation and vibration as pipeline bedding and embedment. Several field trials were used to establish the best gradation and the effectiveness of the procedure.

As clean gravels are dumped into a stockpile, the larger particles tend to roll to the perimeter of the stockpile. Dumping gravels containing 3 inch particles in beside a pipeline created rock pockets in the haunches of the pipe. Experimentation to find the optimum gradation to prevent these rock pockets resulted in the following gradation:

The gradation for optimum flow characteristics is:

Passing No. 200 sieve 5% or less.
Passing No. 50 sieve 25% or less.
Passing ¾-inch sieve 100%.

Basically, the material must have few fines, not much fine sand, and have a maximum particle size of 3/4 inch *(20 mm).*

Bureau of Reclamation – McGee Creek Aqueduct

The results of the first attempt by a contractor inexperienced with saturation and vibration is shown in Figure 6. There were too many fines in the soil used as evident by the crust on the surface. The contractor ordered the material appropriately with 5% fines or less. However the aggregate processing plant sent the wrong material. Once the correct soil was being used, he successfully completed the job.

Figure 6 Too Many Fines for Saturation and Vibration

SUMMARY

Proper buried pipeline installation relies on support for the pipe in the haunch area. This is necessary for all types of pipe material. One method of obtaining haunch support is by using cohesionless soils and providing compaction by using saturation and internal vibration (jetting and vibrating). The method uses internal concrete vibrators and enough water to lubricate the soil particles.

The method has two significant advantages: (1) the soil in the pipe haunch area can be effectively compacted to a high density, and (2) the compacted lift thickness can be several feet thick, limited by only the length of the vibrator.

The disadvantages of saturation and vibration are floating the pipe if too much water is used and sometimes a trial test section is necessary to arrive at the right combination of water, equipment, and procedure.

Denver Water has been utilizing this method since the 1960s. US Bureau of Reclamation has been using the method since the 1950s and at one time saturation and vibration was the only acceptable compaction method for pipe embedment.

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Sayreville Relief Force Main: 10 Years of Monitoring and Proactive Management

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Abstract

Proactive management, monitoring, and rehabilitation of pipelines are common phrases used in the water and wastewater industry today. Over the last 10 years, tools and techniques have rapidly evolved, providing utilities with a plethora of options for pipeline assessment and management. Because proactive management and monitoring are only recently being widely implemented across the industry, few long-term case studies documenting the success of these practices are available. This paper will discuss the monitoring and proactive management of the 102-inch Sayreville Relief Force Main, which has been actively monitored and routinely inspected for the last 10 years. When the Sayreville Relief Force Main experienced its second failure in March of 2003, many of the common techniques for condition assessment and asset management were just being introduced to the industry. Due to known deficiencies associated with Interpace PCCP manufactured in the 1970s, aggressive soils, and an environmentally sensitive surrounding area, the force main's owner and operator, Middlesex County Utilities Authority (MCUA), began a program of inspection, assessment, and active monitoring. Over the 10-year management period, monitoring evolved from a surface-mounted sensor (SMS) system to an acoustic fiber optic (AFO) monitoring system, and the assessment techniques grew from visual and sounding inspections and electromagnetics to include sonic/ultrasonic testing, and advanced structural analysis. Through this program, MCUA has proactively rehabilitated deteriorating pipes, preventing catastrophic failures. A study of the evolution of the program for the Sayreville Relief Force Main can serve as a resource for other water and wastewater utilities, who will benefit from the lessons learned and the advantages gained over 10 years of proactive management.

Introduction

MCUA provides wastewater treatment services for over 800,000 residents of central New Jersey. One of the largest pipelines in the MCUA system is the 102-inch Sayreville Relief Force Main, which extends 18,700 feet from the Sayreville Relief Pump Station to the Edward J. Patton Water Reclamation Facility. When the Sayreville Relief Force Main experienced its second failure in March of 2003, many of the common techniques for condition assessment and asset management were just being introduced to the industry.

The Sayreville Relief Force Main comprises 102-inch prestressed concrete cylinder pipe (PCCP) – embedded cylinder type (ECP). This particular design includes an inner concrete core, a thin steel cylinder, an outer concrete core, high-strength steel prestressing wire, and a mortar coating. The prestressing wire is wrapped helically around the concrete core to hold the concrete in compression when internal and external loads are applied. The concrete cores and prestressing wire provide the structural strength for the pipe while the steel cylinder provides water tightness and the mortar coating protects the prestressing wire wraps. The ECP in the Sayreville Relief Force Main was manufactured by Interpace Corporation (Interpace) in the late 1970s. The deficiencies of Interpace pipes manufactured in the 1970s are well documented and were most particularly related to the manufacture of the prestressing wire.

The 2003 failure in the Sayreville Relief Force Main was attributed to acidic soils that deteriorated the mortar coating of the pipe and lowered the pH of the coating. No longer protected by the alkaline environment, the wire was exposed to a corrosive environment. Additionally, the Interpace 8-gage, Class IV prestressing wire used in the Sayreville Relief Force Main is known to be particularly susceptible to hydrogen embrittlement and poor torsional ductility. This combination led to sudden, brittle breaks in the prestressing wire wraps. Broken prestressing wire wraps ultimately led to the failure of the pipe.

The first investigations of the Sayreville Relief Force Main focused on identifying any pipes in immediate danger of failure as well as pipes with low to moderate levels of damage. The investigative techniques included visual and sounding inspection to identify pipes in a state of incipient failure, electromagnetic inspection to identify pipes with broken prestressing wire wraps, and soil and groundwater testing. Following the internal inspections, a continuous acoustic monitoring system was installed to track prestressing wire wrap breaks in near real time.

Since MCUA implemented the assessment program in 2003, the Sayreville Relief Force Main has been inspected in its entirety three (3) times, 2003, 2008, and 2013, with a number of shorter, targeted inspections performed in the intermediate years. Since that time, the original inspection techniques have been improved and new tools were also introduced.

Evolution of Inspection Tools and Techniques

Since the failure in 2003, numerous tools and techniques have been used to evaluate the Sayreville Relief Force Main. The following sections detail how these inspections have been conducted in the Sayreville Relief Force Main and how they have evolved since the first comprehensive internal inspection in 2003.

Visual and Sounding Inspection

The visual and sounding inspection methodology has remained largely unchanged over the 10 years that the condition assessment program for the Sayreville Relief Force Main has been in place. In fact, Openaka Corporation, Inc. (Openaka), now owned by Pure Technologies U.S. Inc., refined the current visual and sounding techniques for PCCP in the early 1990s.

Visual and sounding inspections are used to detect pipes in a state of incipient failure. During the visual and sounding inspections, the interior of the Sayreville Relief Force Main was inspected for cracks, spalls, and other signs of distress. Additionally, a steel rod was used to strike the interior surface of the pipes to detected hollow areas. It has been shown that longitudinal cracks at the springline with carbonate staining, hollow areas, and especially a combination of the two, can be indicators that a pipe is in a state of advanced distress (Lewis and Wheatley). Figure 1 shows an inspector sounding during the 2013 internal inspection.

Figure 1. Inspector Sounding the 102-inch Force Main

Electromagnetic Inspection

While a visual and sounding inspection identifies pipes in a state of incipient failure, it cannot detect pipes with minor to moderate levels of distress. To complement the visual inspection, electromagnetic inspections of the Sayreville Relief Force Main are also performed. An electromagnetic inspection is a nondestructive method used to evaluate the current condition of the prestressing wire wraps. Pipes in a state of incipient failure typically have a large number of broken wire wraps.

Electromagnetic inspections of the Sayreville Relief Force Main began in 2003. The theory behind this technology was that a varying electromagnetic field is applied to the helically-wrapped prestressing wire. Discontinuities in the prestressing wire (i.e., broken wire wraps), alter the field (Lewis and Wheatley). Changes in the detected field are measured and can be used to locate and quantify distressed regions in PCCP.

The theory behind electromagnetic inspection has not changed in the 10 years that the condition assessment program for Sayreville Relief Force Main has been in place. What has been improved is the configuration of the inspection tool. The detectors in the original tool were oriented in such a way that any non-uniform pipe properties were detected. The newest tools have adjusted the configuration of the detectors so that they specifically look for changes in the prestressing wire. Additionally, as more pipelines were inspected and results were validated, analysis of the inspection results was refined and improved.

Continuous Acoustic Monitoring

Large areas of broken prestressing wire wraps can lead to catastrophic failure of a PCCP. Unfortunately, PCCP does not fail in a uniform manner and large amounts of damage can occur in a relatively short period of time. To monitor the Sayreville Relief Force Main in between manned internal inspections, MCUA opted to install continuous acoustic monitoring equipment to detect wire wrap breaks as they occurred in near real time.

Because the Sayreville Relief Force Main conveys wastewater, a surface mounted sensor (SMS) system was chosen to continuously monitor the pipeline for wire wrap breaks. The first monitoring system, which was installed in January 2004 following the first internal inspections, consisted of surface mounted sensors attached to accessible appurtenances on the exterior of the force main. These sensors detected the wire wrap breaks and transmitted the results to a central computer. MCUA personnel received notifications of wire wrap breaks via e-mail (Fitamant, Lewis, et al.).

Following the 2008 and 2013 inspections, new acoustic fiber optic (AFO) systems were commissioned in the Sayreville Relief Force Main. Unlike the SMS system, fiber optic cable was installed along the entire length inside the force main. Wire wrap breaks can be recorded at any point along the cable and the information about a particular break is transmitted back to a processing computer along the pipeline. Figure 2a and Figure 2b show internal and external views of a splice point at a manhole in the current MCUA AFO system. At splice points, runs of fiber optic cable from different portions of the pipeline are connected to create one continuous system.

Figure 2a. Internal AFO Splice Point at MCUA

Figure 2b. External AFO Splice Point at MCUA

Although notifications are still sent to MCUA via e-mail, all wire wrap break data is also accessible on a website that allows MCUA personnel to view the wire wrap break history for the entire force main.

Structural Modeling

As noted previously, when a visual and sounding inspection detects a pipe with a hollow area and longitudinal cracking, it typically indicates a pipe in a state of incipient failure. It is then a fairly straightforward decision to repair pipes with this level of distress. When the results from an electromagnetic inspection and acoustic monitoring are considered, the question becomes at which level of prestressing wire damage does action need to be taken to mitigate the risk of catastrophic failure.

Structural modeling can be used to evaluate the condition of a PCCP design with varying numbers of broken prestressing wire wraps. To evaluate the results for pipes with minor to moderate levels of distress, MCUA opted to perform structural modeling to assist in management decisions for the Sayreville Relief Force Main. This allowed MCUA to schedule future rehabilitation work rather than needing to perform immediate repairs because a pipe had a very high risk of failure.

Following the 2003 inspections, two-dimensional finite element modeling was used to investigate the results of the electromagnetic inspection and the AFO monitoring. The PCCP was modeled as a two-dimensional beam on an elastic foundation. The cross section of the pipe was transformed into an equivalent concrete section and the radial stiffness of the pipe was used as the spring stiffness of the elastic foundation. The compression applied by the prestressing wire, the internal water pressure, and any external loading were combined via the principle of superposition and applied to the beam. The beam deflection calculated in the analysis could then be equated to the stress generated in the concrete. That level of stress was compared to the compressive strength of the concrete to determine when visible cracking would occur (Fitamant et al.).

Since the initial structural evaluations, advancements in computer processing capabilities led to the use of comprehensive three-dimensional, nonlinear finite element modeling of the 102-inch ECP designs used in the Sayreville Relief Force Main. In these evaluations, the PCCP design could be modeled as a pipe, with each of the cross section components acting as a layer in a composite element. The pipe that is ultimately modeled is a collection of thousands of individual elements. Figure 3 shows the hoop stresses developed in the prestressing wire layer of a 102-inch PCCP with 35 broken wire wraps.

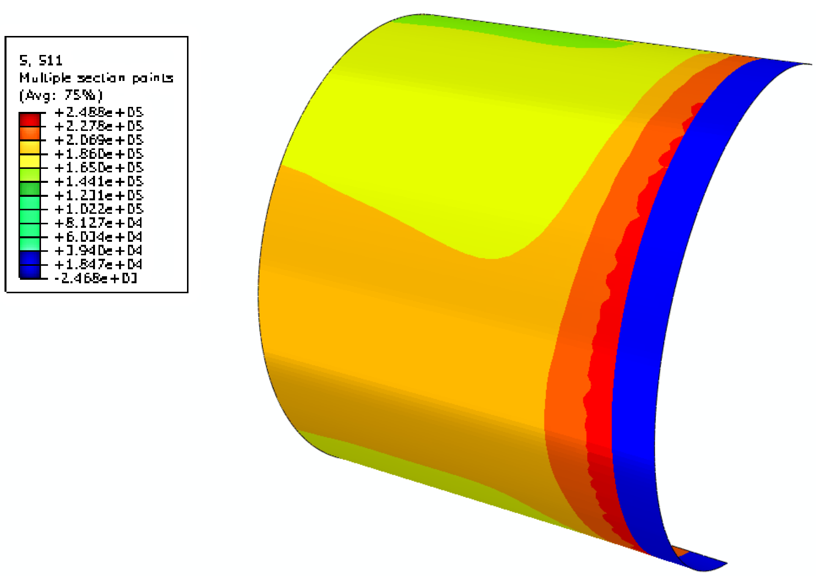

Figure 3. Hoop Stresses Developed in the Prestressing Wire Layer

The internal pressure and external loads are applied to the modeled pipe and the resulting stress and strain in the pipe wall is captured and analyzed. To simulate damage, prestressing wire wraps are removed from the model and the resulting stresses and strains are measured. The American Water Works Association (AWWA) C304 cracking limits for the concrete core as well as the yield and ultimate tensile strength limits of the steel components of the pipe are used as performance limits for the pipe. When pipes exceed certain performance limits, the results are reported and MCUA can make the appropriate management decisions.

Sonic-Ultrasonic Testing

Although not one of the original technologies used to inspect the Sayreville Relief Force Main, sonic/ultrasonic testing, also known as impact echo, was implemented as part of the inspection program in 2012. Sonic/ultrasonic testing has been used by MCUA to inspect specific pipes in the force main that had large electromagnetic anomalies indicative of broken prestressing wire wraps. In this testing, impacts are made to the pipe wall and the resulting sonic waves are measured. The velocity of the reflected wave, as well as the frequency and amplitude of the following reflections (echoes) are measured to detect cracking in the concrete core as well as any delaminations in the concrete core (Analytical Engineering, Inc.). This technology was used by MCUA to confirm the results of an electromagnetic inspection, particularly for pipes where there was not significant enough damage to cause hollow areas and longitudinal cracks.

Outcome of the Assessment Program

Following each internal inspection of the Sayreville Relief Force Main, MCUA received recommendations regarding which pipes were in need of repair. The pipe list, based on the results from the internal inspections, included pipes with hollow areas and longitudinal cracks, as well as pipes that had a significant number of broken prestressing wire wraps, when compared to the structural evaluations.

In addition, continuous monitoring of the Sayreville Relief Force Main has allowed MCUA to identify actively deteriorating pipes and plan for their rehabilitation. While the results are provided via the website and e-mail updates, quarterly reports, summarizing the results of the monitoring period and evaluating the new broken wire wrap estimates on each of the pipes with damage, are also provided to MCUA to assist in rehabilitation decisions.

As a result of the assessment program, MCUA has proactively rehabilitated 15 pipes using post-tension tendon repairs. These repairs involved excavating the pipe in question and wrapping seven-strand tendons around the outer circumference of the pipe. These tendons act to keep the concrete core in compression even with broken prestressing wire wraps. The pipes are then encased in concrete to provide an additional layer of protection. Figure 4 shows a tendon repair of a pipe in the Sayreville Relief Force Main.

Figure 4. Tendon Repair of 102-inch PCCP

Like all of the tools and techniques in the Sayreville Relief Force Main, the rehabilitation methodology is also beginning to evolve. Many of the previously repaired pipes were located in relatively isolated areas where excavation would not cause significant disruption to the surrounding community. Recently, the use of carbon fiber reinforced polymer (CFRP) repairs as a method of rehabilitation has been considered for the Sayreville Relief Force Main. This method of internal repair will allow MCUA to repair pipes in areas that are difficult to excavate.

Conclusions

MCUA has not experienced a catastrophic failure of the Sayreville Relief Force Main in the 10 years that the proactive management program has been in place. Core techniques and technologies, including visual and sounding inspection, electromagnetic inspection, acoustic monitoring, and structural evaluation, have been used throughout the program to provide MCUA with actionable results. Most of these techniques have evolved over the 10-year period, providing MCUA with increasingly higher-resolution results. Through this program, MCUA has proactively rehabilitated deteriorating pipes, mitigating the risk of catastrophic failures.

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Incorporating GIS-Based Structural Evaluation Tools into Pipeline Asset Management

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Abstract

This paper presents the recent enhancements to Tarrant Regional Water District's (TRWD's) geographic information system (GIS)-based structural evaluation tool, how this tool is used to improve asset management capabilities, and lessons learned from development of TRWD's GIS for use in pipeline asset management. TRWD now utilizes a GIS and a GIS-based tool capable of performing structural evaluation of non-distressed pipes, failure margin analysis and repair prioritization of each distressed pipe, and pipeline diagnostics as a part of their asset management program. TRWD recently updated their GIS and enhanced their existing tool to be able to identify pipes that have high rates of distress progression, add flexibility for projection of distress into the future, and plot analysis results for easy visualization. TRWD periodically performs electromagnetic inspection of their pipelines, and the GIS-based tool provides the ability to update the risk of failure of distressed pipes based on the latest electromagnetic inspection results and identify the pipes at highest risk of failure for remediation. A GIS and customized GIS-based structural evaluation tools provide TRWD with the ability to effectively aggregate pipeline data and perform the analyses needed to evaluate the structural safety of each section of the pipeline system and failure margin and repair priority of individual distressed pipes.

INTRODUCTION

An asset management program that combines regular inspections, forensic analysis, and failure margin analysis will result in utilities effectively allocating scarce resources to high risk sections of the pipeline system while maintaining an acceptable level of risk throughout their system. Utilizing a GIS-based approach to asset management provides a comprehensive view of the pipeline system in the capital improvement planning process and allows for the use of GIS-based tools that perform structural evaluation and failure margin analysis. Repairs of distressed prestressed concrete cylinder pipe (PCCP) can be prioritized using the GIS-based Structural Evaluation Tool (SE-Tool) which is capable of calculating the failure margin of PCCP. SE-Tool can quantify the margin of failure of distressed pipes and prioritize repairs based on the latest pipeline inspection results. Structurally deficient sections of pipeline can be identified using SE-Tool which is capable of evaluating the structural safety of PCCP based on individual pipe properties and the applied internal and external loads.

In order for the GIS to be used effectively for asset management, the GIS pipeline model must contain detailed design, inspection, and repair data for each individual pipe. The pipeline data should be verified against original construction records and against inspection records when available. The pipeline data should be updated with maintenance activities, repairs, and new inspection results in order for the GIS to accurately reflect the state of the pipeline.

TRWD's asset management program concentrates repairs on the pipes at highest risk of failure and spreads pipeline maintenance costs over a number of years to ease funding issues. TRWD has been inspecting their PCCP lines annually since 1998 and currently uses SE-Tool to evaluate the risk of pipe failure and prioritize pipe repairs. Based on their use of the GIS as an asset management tool, TRWD recently embarked on a program to verify their GIS data and improve their available pipeline analysis tools. TRWD's overall asset management approach, the enhancements provided in SE-Tool, and how the tool fits into their management approach will be discussed.

STRUCTURAL EVALUATION TOOL ENHANCEMENTS

TRWD has been using a GIS-based tool capable of evaluating the structural safety of PCCP, calculating the risk of failure of distressed PCCP, and presenting the results within the GIS as discussed in Nardini et al. (2013). Simpson Gumpertz & Heger Inc. (SGH) and de maximis Data Management Solutions, Inc. (ddms) have created SE-Tool, which includes significant improvements to the original tool as well as additional features. The discussions below focus on the enhancements made to the tool without reiterating the original development.

Failure Margin Analysis using SE-Tool

The purpose of performing failure margin analysis is to determine repair priorities of distressed pipes based on the number of wire breaks estimated by nondestructive inspection technologies such as electromagnetic (EM) inspection, pipe design data, and applied loads. Repair priorities are calculated using the risk curves technology, which is based on structural analysis calibrated and verified by hydrostatic pressure testing of pipes with broken wires to failure, nonlinear finite element analysis, and external inspection of pipes (Zarghamee et al. 2003 and Ojdrovic et al. 2011). SE-Tool calculates failure margins and repair priorities of distressed pipes based on the measured number of wire breaks at the time of EM inspection, the measured number of wire breaks plus measurement uncertainties, and the measured number of wire breaks plus measurement uncertainties and future progression of distress.

Risk curves are developed considering wire failure due to corrosion or hydrogen embrittlement. There may be some sections of pipeline that are more prone to hydrogen embrittlement, resulting in higher residual strength than pipes with corrosion type wire failures. The risk of failure of distressed pipes in such areas can be evaluated considering embrittlement type failure with residual prestress.

Failure margin analysis results are presented in graphical and tabular format. Visualization of the distressed pipe repair priorities in ArcMap is shown in Figure 1. Repair priorities for the three values of effective number of wire breaks are stored on three layers in ArcMap for easy visual comparison. Example risk curves showing the repair priority calculated for each of the three values of effective number of wire breaks is shown in Figure 2.

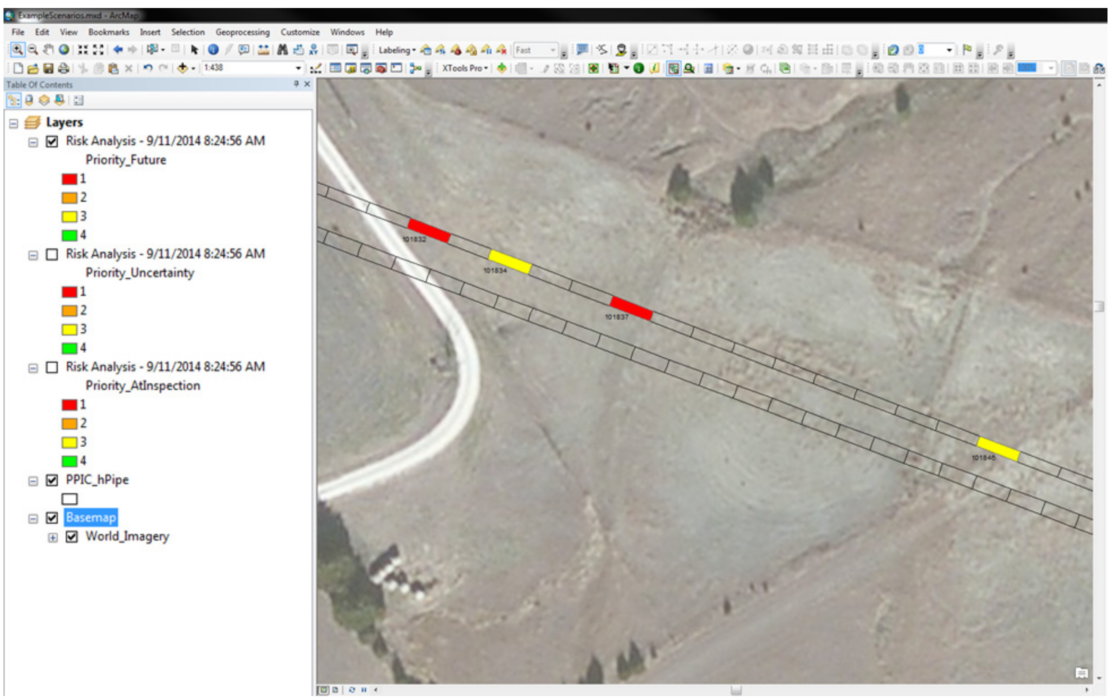


Figure 1 – View Repair Priorities in ArcMap

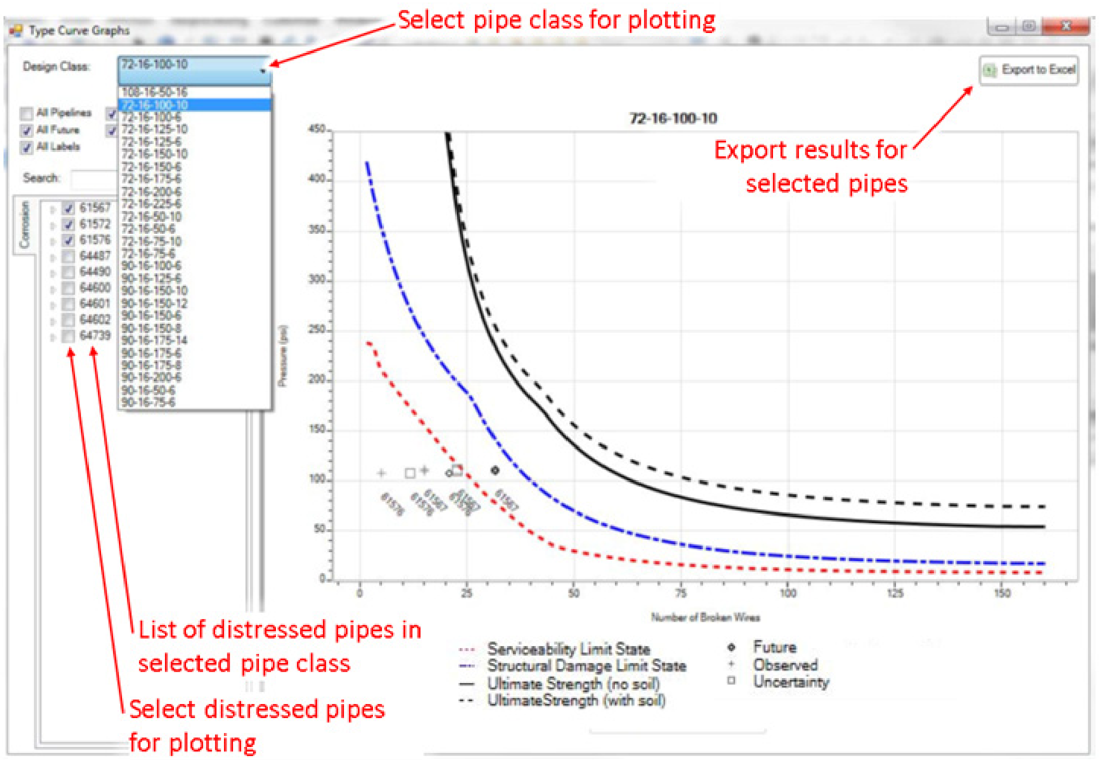


Figure 2 – Example Risk Curves

Structural Evaluation using SE-Tool

SE-Tool calculates demand-to-capacity ratios for pipes without distress under the given loading conditions for several limit states. The structural adequacy of a given pipe is evaluated by calculating the ratio of the specified maximum pressure (P_{max}) over the critical pressure ($P_{critical}$) corresponding to reaching various limit states defined by AWWA C304 – Standard for Design of Prestressed Concrete Cylinder Pipe. The value of P_{max} /$P_{critical}$ for each pipe is output to the database, and ratios exceeding 1.0 indicate violation of the given limit state. Ratios of P_{max} /$P_{critical}$ along the pipeline can be exported from the GIS database and plotted as shown in Figure 3 to identify areas of potential concern where critical limit states that affect pipeline durability are violated.

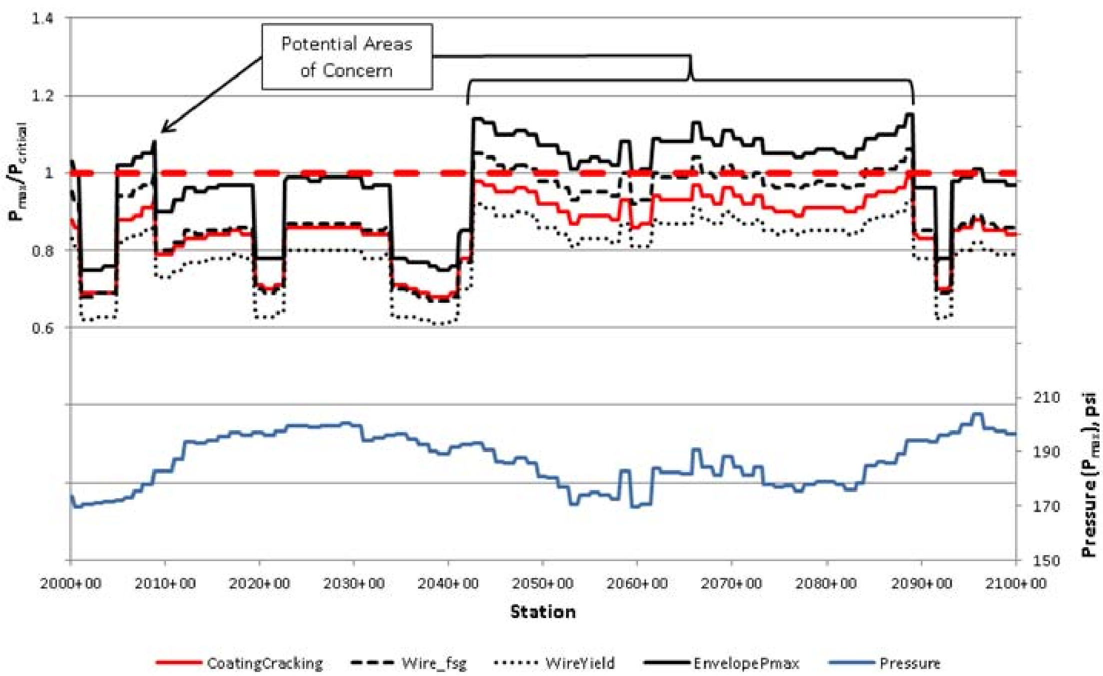


Figure 3 – Example plot of P_{max} /$P_{critical}$ along a portion of pipeline

Pipeline Diagnostics using SE-Tool

Broken Wire Zone Growth Rates. The rate of progression of wire breaks is calculated based on historical wire break data from past EM inspections. To calculate growth rates, the algorithm considers all inspection records on a given pipe and identifies where a specific BWZ is reported in more than one inspection.

Two types of growth rates are calculated: short-term growth rates (between consecutive inspections on a matched zone) and average growth rates (from the first to the last inspection on a matched zone). Where there are two inspection records for a matched zone, the short-term and average growth rates will be the same. Where there are three (or more) inspection records for a matched zone, then two (or more) different short-term growth rates are calculated—one between each pair of consecutive inspections—and the average growth rate is calculated between the first and last inspection on the matched zone. Where a zone exists in only one inspection (i.e., it is not matched to any other zone reported in subsequent inspections) then no growth rates can be calculated and that zone is not shown in any pipeline diagnostic results.

Projection of Future Distress. Pipeline Diagnostics will calculate the mean and standard deviation of the BWZ growth rate for the entire pipeline and for BWZs in each repair priority. The growth rate statistics for BWZs with a given Repair Priority at time of inspection can be used to determine whether BWZ growth rates depend on the level of distress.

Identify Pipes with High Broken Wire Zone Growth Rates. Pipeline Diagnostics will generate a table listing each pipe with high rate of wire breakage between consecutive inspections, along with the corresponding historical average growth rate, most recent growth rate, and the inspection data where rapid growth was observed. The historical average growth rate is calculated as the total change in NBW from the first inspection to the last inspection on the pipe that reported distress in the matched zone, divided by the interval of time between those inspections. The most recent growth rate is the growth rate calculated using the two most recent inspections of a matched zone.

Execution of SE-Tool in ArcGIS

Leveraging the power of ESRI's ArcGIS platform, the SE-Tool is delivered as an add-in for ArcMap. This model provides a conveniently packaged compressed file for delivery that allows for a framework that can be easily shared between users and does not require additional installation programs or registrations. Combining the ESRI software platform, TRWD's asset management system, and the SE-Tool allows for users to analyze, plan, visualize and share the important risk and structural analysis information across stakeholders.

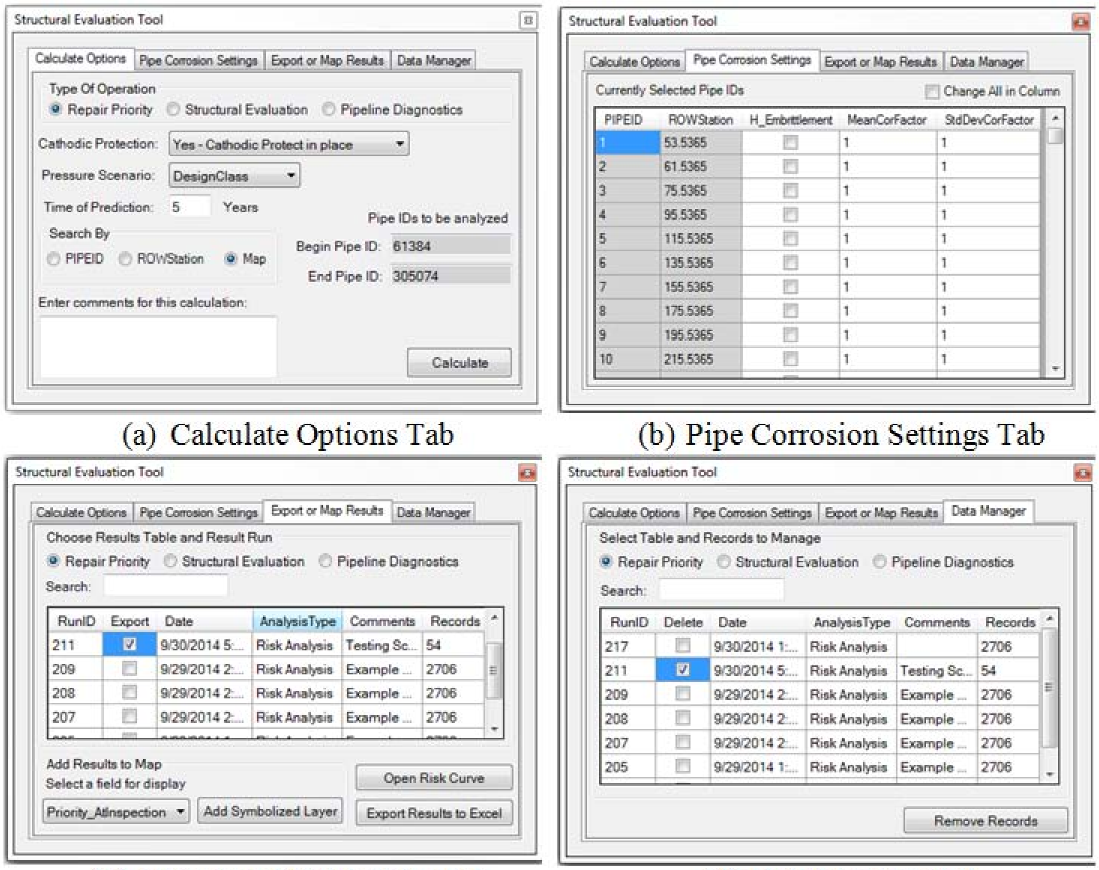
The SE-Tool is home to the Repair Priority, Structural Evaluation, and Pipeline Diagnostics modules. The interface is divided among four main operations: Calculate options, Pipe Corrosion Settings, Export/Map Results, and Data Manger. Within the calculate options tab of the SE-Tool (Figure 4a) the user selects the analysis operation and defines some key variables for the analysis (e.g. cathodic protection, pressure scenario, time period for BWZ growth prediction). Added functionality allows the user to select the pipe analysis range by PipeID, Right-of-Way Station, or by graphically selecting a region within the map.

The Pipe Corrosions tab of the SE-Tool (Figure 4b) is exclusive to the failure risk analysis calculations. This tab allows the user to identify pipes likely suffering from hydrogen embrittlement and to input customized mean and standard corrosion factors for individual pipe segments or the entire analysis.

Users are allowed to export their data in a variety of formats from the Export or Map Results tab (Figure 4c). The SE-Tool allows the user to export tabular results from any of the analysis modules to a Microsoft Excel format. The graphical export available for the failure risk analysis is based off of the three repair priority types: at inspection, future, and uncertainty (Figure 1). Also available for repair priority is the ability to view and export risk curves (Figure 2).

The Data Manager tab (Figure 4d) allows the user the ability to maintain or delete previous analyses from the database. As users are able to export and view previously performed analyses, it is important that users be able to maintain a clear record without needing to have access to the administrative database.

SE-Tool's user interface helps navigate the user through multiple analyses and includes a number of safeguards for data integrity. SE-Tool produces valuable messages and error logs as needed for users to understand any issues encountered.



(a) Calculate Options Tab (b) Pipe Corrosion Settings Tab

(c) Export or Map Results Tab (d) Data Manager Tab

Figure 4 – SE-Tool User Interface

TRWD APPROACH TO PIPELINE ASSET MANAGEMENT

TRWD has pursued three paths to help mitigate failures on the transmission lines which include transient pressure control, cathodic protection, and pipe segment replacement as discussed in Nardini et al. (2013). GIS has played a crucial role in the pipe segment replacement portion of TRWD's pipeline asset management program.

As part of TRWD's asset management plan, an area of the PCCP pipeline is selected to be inspected annually using electromagnetic inspection technology. After each inspection, the GIS database is updated with the results of inspections of each individual pipe segment and SE-Tool is run using the most up-to-date inspection results. Multiple scenarios are run with SE-Tool to see how changing factors, such as the pressure and wire failure mechanism (corrosion vs. hydrogen embrittlement), impacts the repair priority of distressed pipes.

In 2014, TRWD advanced their pipeline asset management program by incorporating a spatial risk model that quantifies hazards and consequences of failure. Repair priority output from the SE-Tool is one of the highest weighted factors in the risk model. Other factors, such as time since last inspection, corrosion data, land use, proximity to utilities, railroads, and highways, have been incorporated into the spatial risk model. The output includes priority rankings, risk factors, and pipeline stationing. These results are placed into a table for analysis and review.

Once pipe segments have been chosen and replaced, a detailed forensic study is done on each segment. It is important to determine which type of damage is found (corrosion or embrittlement), number of wire breaks, and any cracking or issues with the mortar lining/coating. The findings are all archived within GIS to use in the risk model for future assessment. Results from the forensic investigations, including the number and locations of wire breaks observed on each pipe, are compared to the EM inspection results and used to aid in improvements to wire break estimates in the future to the extent possible. This complete pipeline asset management cycle (Figure 4) has helped give TRWD an understanding of past events, the current state of the system, and what we may encounter in the future.

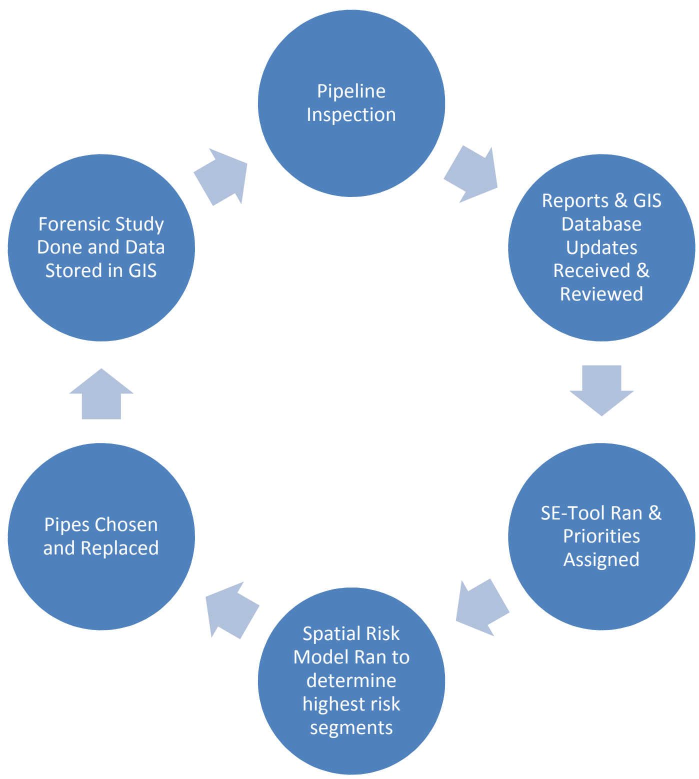


Figure 4 – TRWD Inspection/Replacement Cycle

Selection of Pipeline Sections for Inspection

The GIS is used to select sections of pipeline for inspection. For planning and budgeting purposes, TRWD operates on a three year outlook for all future pipeline inspections. In general, TRWD inspects approximately five miles per year, though it may not necessarily be continuous. Since 1998 TRWD has inspected all 164 miles of PCCP, with some areas having multiple inspections, and now has an accurate map of the pipeline segments and a general understanding of pipe deterioration rates in some areas. Many factors are considered when selecting pipeline sections for inspection, such as:

- SE-Tool structural evaluation results.
- Consequences of failure.
- Historical areas of concern/past failure locations.
- Time since last EM inspection.
- High pressure zones.
- Corrosivity data including pipe-to-soil potential measurements

Selection of Pipes for Rehabilitation

TRWD's typical method of rehabilitation is replacing individual distressed pipes. TRWD generally replaces ten to twenty pipes per year, so prioritizing which pipes to be replaced is crucial. To begin the selection process, the latest inspection results are incorporated into the GIS and the SE-Tool is run. As mentioned, multiple scenarios are run to account for different pressures, wire failure types, time into the future, etc. All of the different scenarios are then incorporated into TRWD's spatial risk model for further evaluation.

To determine the highest risk areas and narrow down the selection for replacement the following factors are considered in TRWD's spatial risk model:

- SE-Tool repair priority output. Figure 5 shows color-coded repair priorities for each PCCP.
- Consequences of failure - land use (urban vs. rural), damage to third party utilities (natural gas, electricity, telecommunications, water, etc.), railroads, roads (highway vs. local street), water loss, downtime, damage to environment, and costs
- Historical data - areas of previous damage and what type of damage was found, proximity to past failures, time since last EM inspection, age of pipe, and known areas of past issues (over-pressured zones, none or bad shorting straps, excess impressed current in early years, etc.).
- Distress data - high pipe-specific rate of wire breakage, wire break concentrations near segment ends, and proximity to high priority pipes.
- Pipeline operation data - cathodic protection data, and higher pressure zones.
- Environmental data – soil types

Using the GIS, SE-Tool, and the spatial risk model, TRWD can quantify likelihood and consequences of failures and select the list of pipes to be repaired/replaced.

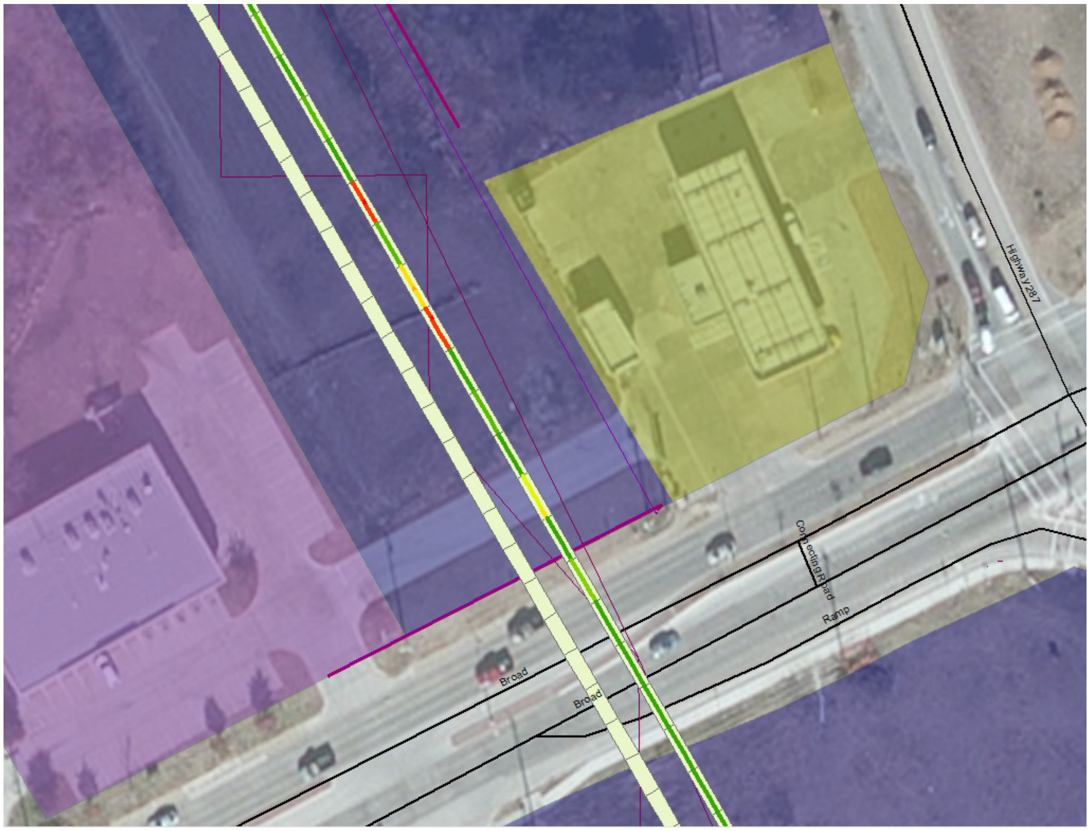


Figure 5 – Zones of Consequence of Failure and Color-Coded Repair Priorities in ArcMap

Data Verification Efforts

After TRWD has replaced segments of distressed pipes, extensive forensics are performed on each individual segment. TRWD records important attributes including the date of replacement, if the replacement was routine or a failure, and the type of damage found on the wires. The inner concrete core and mortar coating are checked for deterioration, cracking, or any other abnormalities. Samples of the inner concrete core are also taken and sent to a lab for further evaluation. The mortar coating is removed to expose the wire to determine if the failure mechanism is corrosion, embrittlement, or a mix. All wire breaks are counted and break positions are measured from the end of the joint. After the forensic study is complete, the data is stored in the GIS to be used in the future.

CONCLUSIONS

- GIS is a dependable repository of pipeline assets. Data in the GIS should be pipe-specific, verified, and maintained to accurately represent the current state of the pipeline and provide the information needed for asset management.

- GIS and GIS-based tools such as SE-Tool can be used to select sections of pipeline for inspection, to identify pipes that are structurally deficient, to identify the failure risk of distressed pipe and their repair priority, and to select pipes for repair or replacement.

- GIS-based SE-Tool and the spatial risk model provide data needed for pipeline asset management by evaluating (1) likelihood of failure (based on evaluation of structural adequacy and failure margin analysis of distressed pipes with broken wires) now and in the future considering all uncertainties, and (2) consequences of failure.

- GIS-based tools such as SE-Tool can be economically incorporated into an existing GIS that contains pipe-specific data

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Structural Integrity of Damaged Cast Iron Pipelines and Identifying When Damaged Pipes Should be Repaired or Replaced

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Abstract

The majority of cast iron (CI) pipelines were installed in the United States in the early to mid-20th century. Most of these pipes are in a sustainable working condition; however, any failure of these pipelines could be catastrophic due to the social, political, and environmental impacts of the failure. To facilitate better decision making for capital expenditures and to address the risk associated with these assets, the failure mechanisms of CI pipe must be understood. In addition, for planning and capital budgeting, it is important to determine the current condition of a deteriorated pipeline and its risk of failure. In this study, the structural evaluation of damaged CI pipes is discussed considering corrosion of the pipe (the most common failure mode of CI pipe). This paper provides a clear and concise picture of how to determine the current condition of damaged CI pipelines. This will allow decision makers to enhance their asset management strategy based on inspection results and advanced computational modeling.

Introduction

Cast Iron (CI) pipes have been used as pressure transmission mains for gas, water, and wastewater since the 19th century. Manufacturing of CI pipes ended in 1970; however, many old pipeline systems style have these pipes in service, with the average age being more than 75 years old. Based on the an American Water Works Association (AWWA) survey in 2002, 38.8 percent of water mains and 11.9 percent of force mains in the United States were built using CI [Water Main Inventory – AWWA Water Stats 2002 Distribution, US Force Main Inventory - Guidelines for the Inspection of Force Mains (2010)]. Therefore, CI pipes were extensively used in the United States and it is important to understand the behavior and failure modes of cast iron pipes.

Makar et al. (2001) discusses the modes and causes of pipe failures that have been encountered during a three year investigation by the National Research Council Canada. In this study, they separated the modes of failure based on the pipe diameter, considering small diameter (less than 380 mm), mid-diameter (380 mm to 500 mm), and large diameter (greater than 500 mm) pipes. This study concluded that failure modes vary depending on the diameter of the pipe. For small diameter pipes, bell splitting at the top of the pipe and circumferential cracking at the middle were the primary failure modes. In large diameter pipes, bell shearing and longitudinal splitting were most common. For mid-diameters pipes, spiral cracking and corrosion pitting were the frequent failure modes. Smaller diameter pipes have lower water pressures, which makes them more susceptible to longitudinal bending failures. Circumferential cracking is typically caused by bending forces applied to the pipe. Bending stress is often the result of point loads from poor bedding condition, soil movement, or thermal contraction. Soil movements producing tensile forces on the pipe and cause a simple tensile failure.

Seica et al. investigated the modes of failure of CI pipes in City of Toronto, considering the amount of corrosion and mechanical properties of the pipe.

Atkinson et al. (2002) investigated the in-service strength degradation of CI water distribution pipes as a result of corrosion. They measured the strengths of 1-meter lengths of pipe extracted from the ground using either the 3- or 4-point bending test. The size of the controlling defect was estimated by visual examination of the fracture surface. They concluded that for small CI pipes (diameter less than 100 mm), the critical pit depth that corresponds to the situation when service stress exceeds the residual strength of the material is approximately equal to 30 percent of the pipe wall thickness. They also correlated the residual strength/pit depth data using loss of section and fracture mechanics approaches.

Makar et al.; (2001) and Rajani and Kleiner; (2001) concluded that the main deterioration mechanism on the exterior of CI pipes is electro-chemical corrosion, with the damage manifesting in the form of corrosion pits. The damage to gray CI is often identified by the presence of graphitization. Therefore, the physical environment of the pipe has a significant impact on the deterioration rate.

AWWA C101-67 proposed an empirical parabola equation for designing CI pipes using the three-edge bearing test. The proposed load-pressure equation was used to calculate the minimum required thickness of the pipe. In the standard, the maximum corrosion allowance was 0.08 inches based on experience of early engineers. This value is conservative and no information regarding the length of the corroded area was considered. Furthermore, Rajani et al. (2000) proposed a methodology to estimate the remaining service life of grey CI mains that have corrosion pits. They used the failure stress equation, which relates the crack length and the fracture toughness, and proposed an empirical equation to consider the effects of a single pit.

To date, there has been no study has been done to determine the effects of randomly distributed corrosion in CI pipe. In this study, the Authors generated several computational models to study the effects of length and depth of corrosion in CI pipes. The results were compared to determine the effect of damage length and damage depth on the strength of CI pipes.

Finite Element Analysis

Assessment of the residual strength of a corroded CI pipe under internal pressure is generally performed using Barlow's analytical equation, assuming corrosion occurs around the circumference of the pipe. This assessment considers less realistic geometries for the defect area and does not evaluate the effects of combined internal and external loadings. Finite element analysis (FEA) is currently the most accurate method for the assessment of corroded CI pipes without requiring extensive testing

A three-dimensional finite element model was built and used to perform the analysis of a CI pipe with various corrosion damage configurations. The FEA model was developed to determine the structural effect of corrosion pitting in a particular CI pipe. Defect areas are modeled and manipulated to simulate the growth of the corroded area. The resulting maximum principal stresses developed in the pipe wall were then compared with the minimum yield strength of the CI to determine the internal pressure required to reach yield.

Material Properties

CI is a brittle material, meaning that the material fails suddenly without any noticeable physical changes prior to failure. ASTM A-48 specification lists seven (7) classes of gray CI ranging in tensile strength from 138.9 MPa to 413.7 MPa. The compressive strength of gray CI is approximately three (3) times greater than the tensile strength. Based on this characteristic, CI is effective at carrying high compressive stresses. Since bending is a combination of tension and compression, the bending strength of CI falls between the tensile and compressive strengths and is usually about twice the tensile strength (Handbook of CI Pipe). The stress-strain behavior of CI exhibits a brittle behavior, with no yield point and an abrupt fracture at failure which is given in (Molnar 2004).

To accurately model the stress-strain relationship of CI pipe, the Authors considered the proposed stress-strain curve from Molnar's paper (2004). Figure 1 shows a schematic stress-strain behavior of the ASTM Grade 35 (ISO Grade 250, EN-JL 1040) gray CI that was used in the computational model.

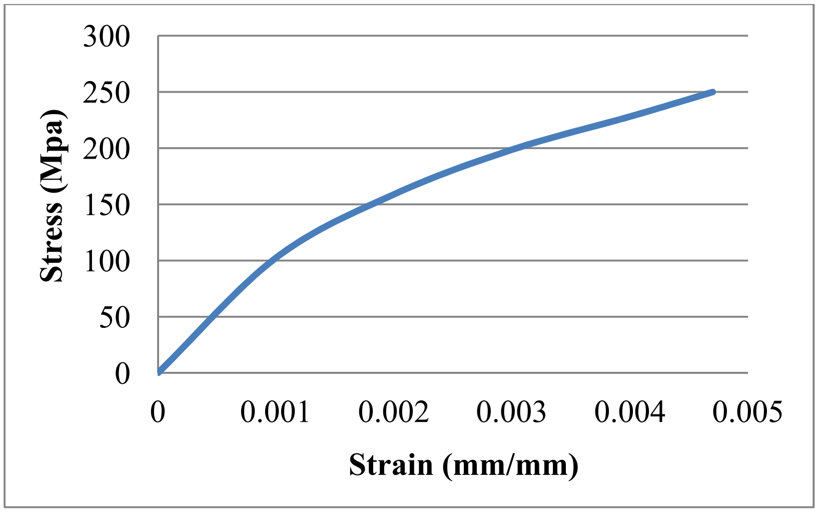

Figure 1. Stress-Strain Curve for Spun Cast Iron

Each CI pipe was modeled using 3D shell elements. Table 1 shows the values of the geometrical and material properties of the CI pipe used for the FEA model.

Table 1. CI Design Specifications

Pipe Parameters	US unit	SI unit
Inside Diameter	24 inches	69.6 mm
Outside Diameter	25.48 inches	647.2 mm
Pipe Wall Thickness	0.74 inches	18.8 mm
Ultimate Tensile Strength	36260 psi	250 MPa
Yield Tensile Strength	23000 psi	159 MPa
Compressive Strength	130000 psi	860 MPa
Young's Modulus	16000 ksi	110 Gpa
Density	450 lb/ft^3	7.2 g/cm^3
Fracture Toughness	436.83 ksi-in$^{1/2}$	480 Mpa-m$^{1/2}$
Poisson's Ratio of Steel	0.24	0.24

The analysis was performed while randomly varying the length and depth of the pitting area. For simplicity, a circular area was considered for pitting, as shown in Figure 2.

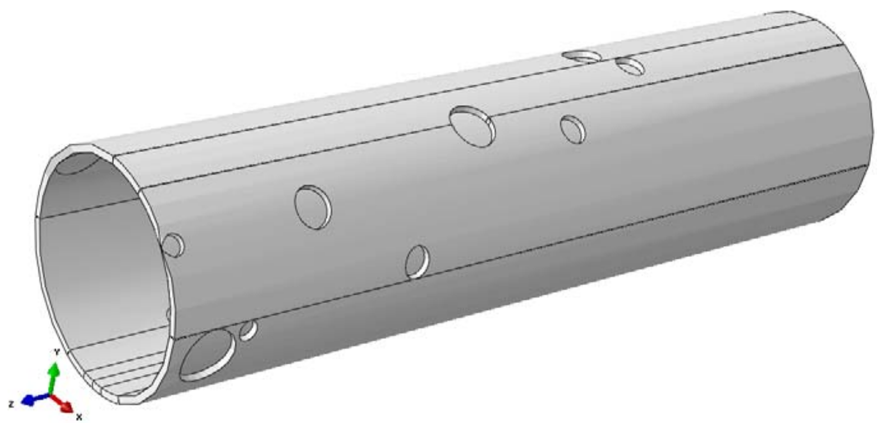

Figure 2. Circular Shape of the Pitting Areas

Seventy-two (72) pipes were modeled considering randomly generated patterns of 10, 20, and 40 pits. Figure 3 shows the location of the pits for one of the modeled pipes, which had 20 pits distributed randomly.

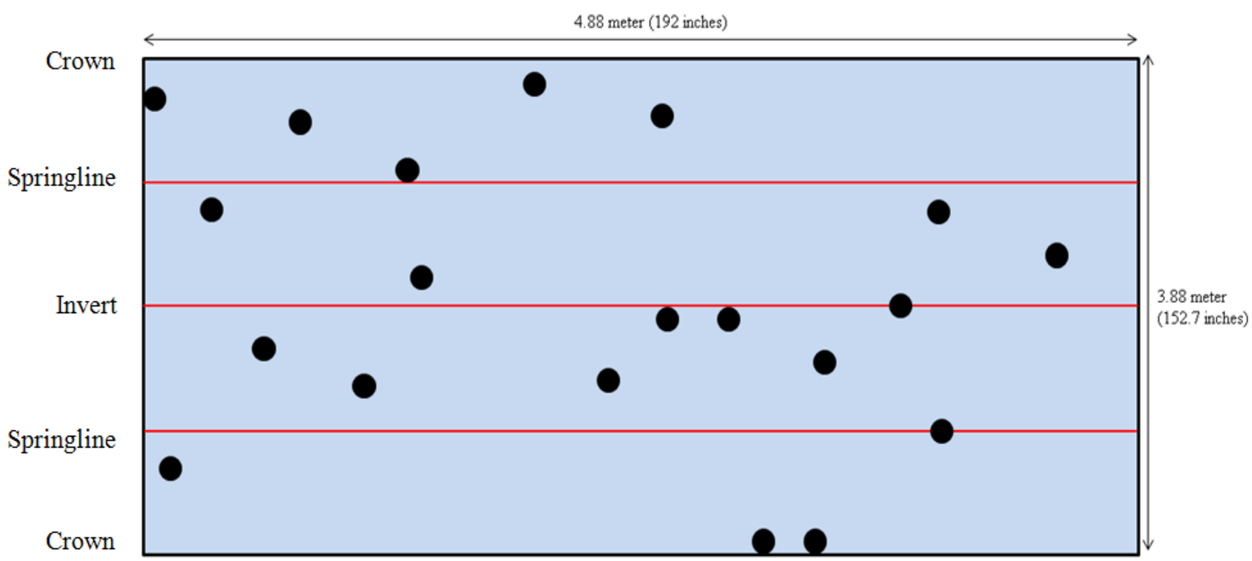

Figure 3. Pits Locations for the Pipe with 20 Pits

Pipes were modeled considering 2.44 meters of earth cover. Density of the soil was considered 1,920 kg/m^3 (120 lb/ft^3). Other loading considered in the FEA model included self-weight of the pipe, live load (traffic load), and internal pressure.

Pit diameters randomly varied from 1 to 10 inches, while the amount of wall loss in the corroded areas varied from 15 percent to 80 percent. To investigate the effect of the size of the corroded area, pit diameters were randomly selected 25.4-76.2 mm, 25.4-152.4 mm, 50.8-101.6 mm, and 101.6-254 mm. Table 2 shows the average pit diameters of each scenario for 10, 20, and 40 randomly generated pits.

Table 2. Average Pit Diameter for Different Scenarios

Scenario	Pit Diameter range	Average Pit Diameter (mm)		
		10 Pit	20 Pit	40 Pit
1	25.4-76.2 mm	50.8	53.34	53.34
2	25.4-152.4 mm	111.76	88.9	91.44
3	50.8-101.6 mm	101.6	124.46	129.54
4	101.6-254 mm	157.48	170.18	170.18

Figure 4 shows an example of the three-dimensional (3-D) mesh used in the model of a CI pipe. Note that to obtain more accurate results for the stresses developed in the corroded portions or pitting zones of the pipe, finer meshes were used in the corroded areas. Figure 4 shows how the pitting zones were considered circular instead of rectangular to prevent stress concentrations at the edges of the damaged zones.

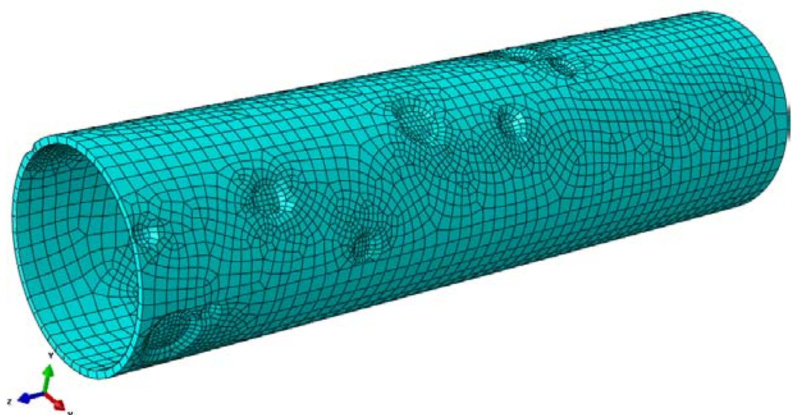

Figure 4. 3-D Mesh of the CI Pipe

To investigate the effects of the severity of corrosion, different level of corrosion of wall loss was randomly generate in six (6) different scenarios: 15-30 percent, 20-40 percent, 20-60 percent, 20-80 percent , 40-80 percent , and 60-80 percent. Because the amount of wall loss in each defect was randomly chosen between the aforementioned ranges, the average amount of considered wall loss in each scenario in this study is shown in Table 3.

Table 3. Average Corrosion Percentage of Wall Loss in Damaged Area for each Scenario

Scenario	Corrosion (%)	Average Corrosion (%) of Pits		
		10 Pits	20 Pits	40 Pits
1	15%-30%	21%	22%	22%
2	20%-40%	27%	29%	30%
3	20%-60%	35%	36%	37%

4	20%-80%	48%	48%	51%
5	40%-80%	59%	60%	61%
6	60%-80%	69%	72%	71%

Results

Figure 5 shows the maximum principal stresses developed in the 24-inch CI pipe with 20 randomly generated pits with diameters of 2 to 8 inches and wall loss of 40-80 percent. The stress shown in the figure is the maximum principal stress, which is the maximum stress developed in each element. To create the FEA curves, failure was considered to occur when the maximum principal stress reached the yield strength of the CI. In Figure 5, color gradients indicate the calculated range of stress for each element in the FEA model.

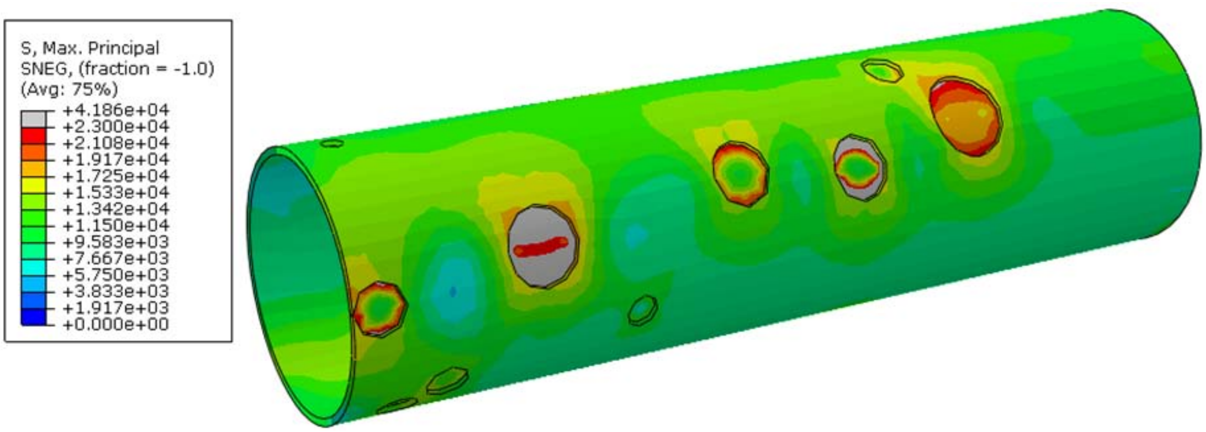

Figure 5. Maximum Principal Stress in 24-inch CIP with Pit Diameter of 2-8 inches and Corrosion of 60-80 Percent

The effect of the total number of defects on the structural integrity of a damaged CI is shown in Figure 6, Figure 7 and Figure 8. Figures 6,7 and 8 show the pressure in which level of the stresses in the damaged pipe reach to yield stresses for 10, 20, and 40 pits, respectively.. Results indicate that for the small diameter pits (range of 25.4-76.2 mm (1-3 inch)) and small number of pits (10 pits) , there are 60% strength reduction in the pipe when the average amount of wall loss increases from 20% to 70%. This strength reduction is more, about 67%, when the average pits diameter increases to 101.6-254 mm (4-10 inch). Figure 7 and 8 show the importance of the total number of defect on structural capacity of a damaged CI pipe. Results indicate that for 20 pits, we have about 55% and 74% strength reduction when the amount of wall loss increase from 20% to 70% for the small diameter pits (1-3 inch), and s large diameter pits (4-10 inch) respectively. We did not see that much changes in strength of the damaged CI when we increased the total number of pits from 20 defects to 40 defects.

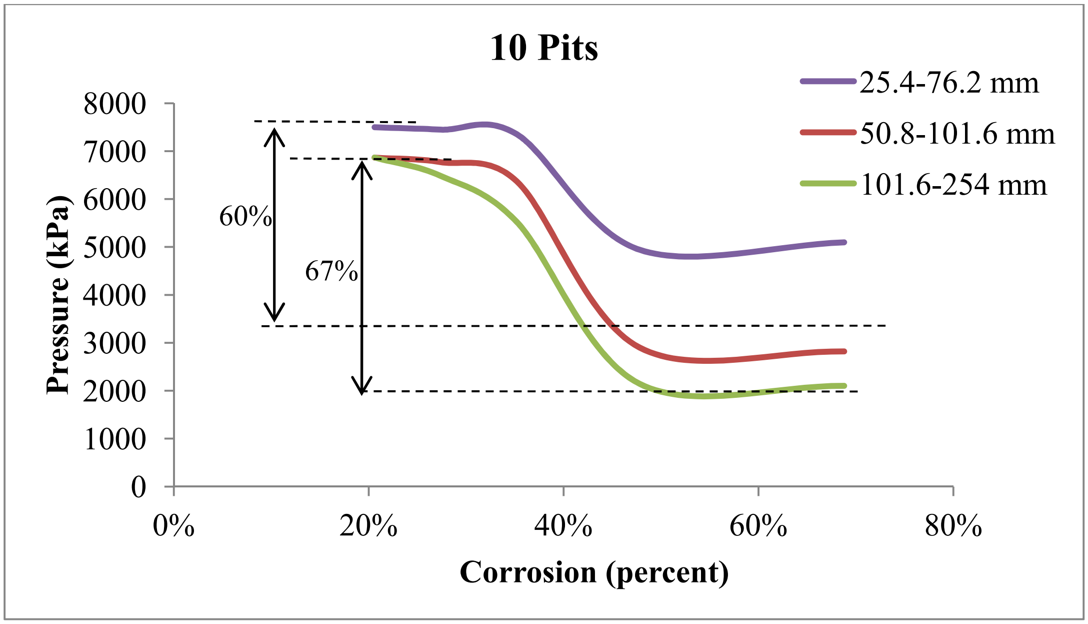

Figure 6. Yield Pressure versus Corrosion Percentage, with 10 Pits in the Pipe

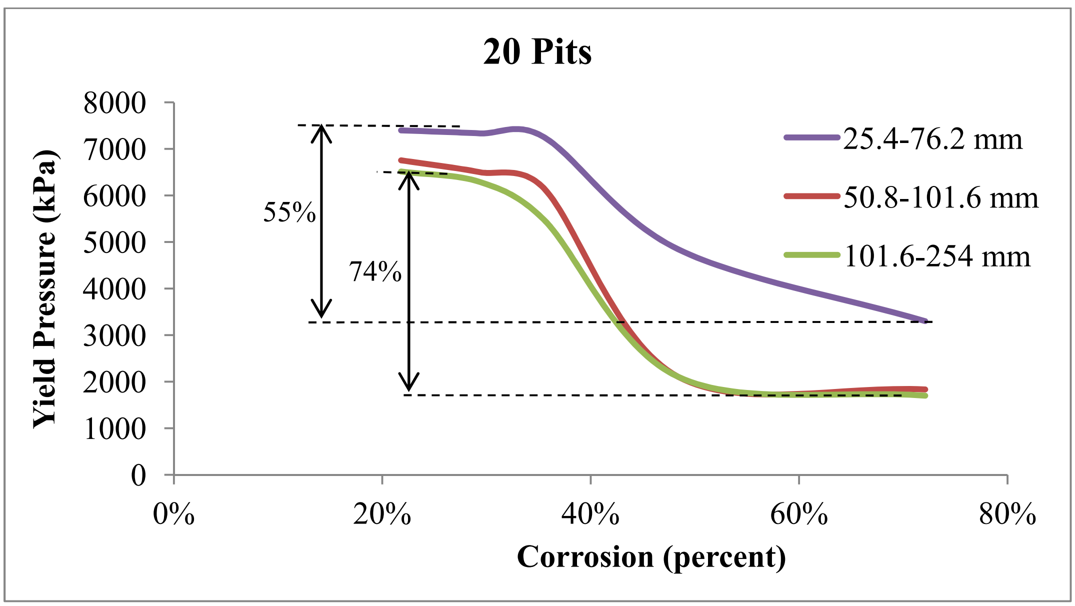

Figure 7. Yield Pressure versus Corrosion Percentage, with 20 Pits in the Pipe

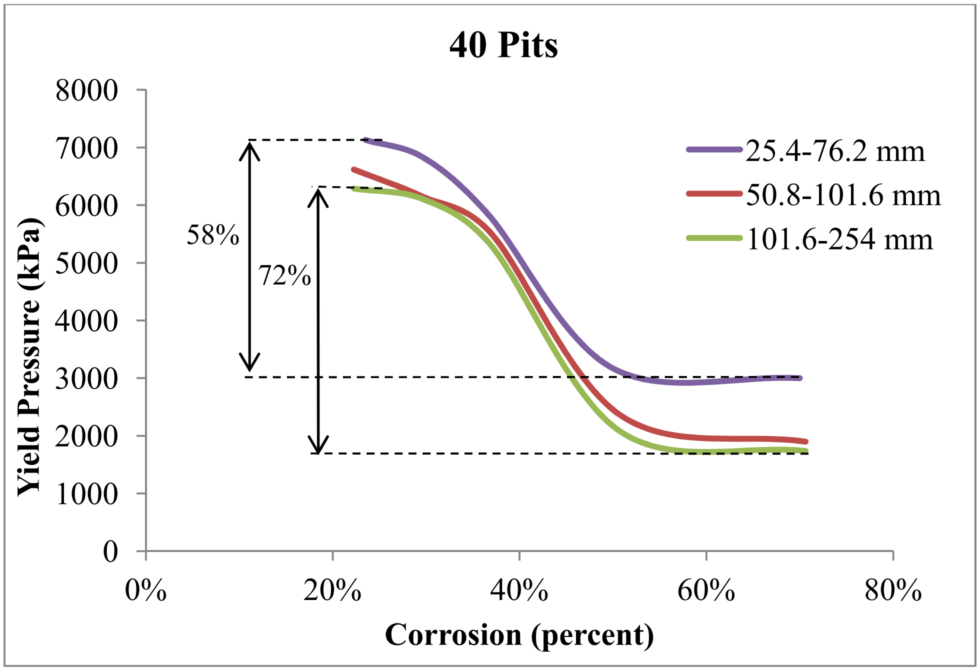

Figure 8. Yield Pressure versus Corrosion Percentage, with 40 Pits in the Pipe

Conclusion

Seventy-two (72) CI pipes with different defect configurations were studied to evaluate the effects of pitting on the load bearing capability of damaged CI pipes. To investigate the effects of number of pits on the structural integrity of CI pipe, 10, 20, and 40 randomly generated pit scenarios were considered. Also, to determine the effect of the size of pitting, pit diameters were randomly changed between 25.4 mm to 254 mm (1 inch to 10 inches). Furthermore, the effect of wall loss was studied in this evaluation varying from 15 percent to 80 percent of the pipe wall thickness.

The results of this study indicated that for small pits (25.4-76.2 mm) and a small number of defects (10 pits), there is about 60 percent strength reduction anticipated when the average amount of wall loss increases from 20 percent to 70 percent. This strength reduction is larger for large defects (4-10 inches), with a greater strength reduction when the number of defects was increased to 20 pits. Also, the Authors noted that if the total number of defects exceeded a certain limit, the increasing number no longer significantly impacted the results. In this evaluation, it was determined that there is not a noticeable change in the strength reduction of a damaged CI pipe when increasing form 20 and 40 total pits. Therefore, the results shows there is a significant strength reduction if the average amount of corrosion was between 40% to 50% and after 60% wall loss the amount of strength reduction become stagnant.

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Water Resources Integration Program Update:
Water Delivery and Operational Flexibility with a 60-inch, 45-mile Pipeline

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Abstract

Over the last several decades, San Antonio has experienced rapid population growth. The San Antonio Water System (SAWS) currently serves more than 1.6 million people in Bexar County Texas as well as parts of Medina and Atascosa Counties, and has over 460,000 water customers. With the integration of the Bexar Metropolitan Water District, regulatory withdrawal limitations, and increasing drought restrictions that have put a strain on the Edwards Aquifer, SAWS has had to increase its water supply and system flexibility. SAWS has been working to meet the increased water demand through a strategy of conservation, reuse, and investments in new water supply resources, and will increase system flexibility with the Water Resources Integration Plan (WRIP). Currently, the Twin Oaks Aquifer Storage and Recovery (ASR) facility in South Bexar County allows SAWS to store excess Edwards Aquifer water from the east side of San Antonio. That water can then be recovered from the ASR during periods of high demand, significantly increasing SAWS operational flexibility. The ASR Pipeline, completed in 2004, is currently the only conduit between the ASR and SAWS' distribution system; moving water into the distribution system during production mode and reversing flow to inject water into the ASR well field in recharge mode. Since future water supply facilities will produce a constant base flow, additional flexibility is needed for continued recharge of the ASR. The WRIP will provide that flexibility. The WRIP consists of approximately 45 miles of 48" to 60" diameter steel pipe and two associated pump stations, extending from the ASR to west San Antonio. The WRIP will be constructed in two phases and will ultimately convey up to 75 MGD of potable water from four different sources: treated water from the Brackish Groundwater Desalination Facility (Wilcox Aquifer), Local and Expanded Carrizo Wells (Carrizo Aquifer), and recovered ASR water (Edwards Aquifer). Similar to the ASR pipeline, the WRIP pipeline will also be used to recharge the ASR well field using reverse gravity flow. The WRIP pipeline will work in tandem with the ASR pipeline to offer operational

flexibility and provide water where San Antonio needs it most. This paper will describe the WRIP and specifically discuss design concepts that allow the WRIP to give SAWS additional flexibility in managing their water supply.

INTRODUCTION

For the last decade SAWS has explored ways to diversify its water supply and lessen its reliance on the Edwards Aquifer. An abundant water source such as the Edwards Aquifer has helped enable San Antonio's growth, but this growth has also had some unintended impacts. In the past, the benefit of the Edwards Aquifer was that new wells were simply drilled in areas of need as population growth necessitated additional water supply. Edwards water rights were relatively inexpensive and easily obtained through lease or purchase from land owners. This method of obtaining additional water created a collection of small distribution systems which were supplied by a nearby well and pump station. This type of system has made it nearly impossible to transfer water from one side of San Antonio to the other. Now, as the city continues to grow, it is expanding beyond Edwards Aquifer production zones and water availability in these areas of high growth has become very critical.

The northwest corner of San Antonio is one area which has recently experienced higher than average growth. This area is supplied with water from SAWS' Anderson Pump Station, but as the population continues to grow, it will eventually exceed the availability of existing water supplies. Future demand combined with SAWS' goal of reducing its reliance on the Edwards Aquifer made this an ideal area to receive new water sources.

BACKGROUND

In 2004, SAWS placed the ASR into operation. The ASR allows SAWS to store Edwards Aquifer water during periods when excess water is available and recover it during dry periods. The ASR is connected to SAWS' Edwards supply and distribution system with a 60" steel pipeline that runs from the Twin Oaks/ASR Facility to three pump stations in eastern San Antonio. This eastern pipeline (ASR Pipeline) regularly moves water from Edwards Aquifer production wells in eastern San Antonioto the Twin Oaks/ASR Facility where the water is then injected into the Carrizo Aquifer. In recovery mode, the ASR Pipeline is used to recover this water and integrate it back into SAWS' distribution system. Because the ASR Pipeline is the only conduit between the Twin Oaks/ASR Facilityto the SAWS distribution system, the Twin Oaks/ASR Facility can only operate in either injection or recovery mode at any given time. This has worked well in the past, but with the development of additional water supplies at the facility, having only one transmission pipeline limitsSAWS' use of these new supplies.

In addition to stored Edwards Water, there are three other current and proposed water supplies that will be treated at the Twin Oaks/ASR Facility:

1. Proposed Brackish Groundwater Desalination Facility (BGD) - Wilcox Aquifer
2. Proposed Expanded Carrizo Program - Carrizo Aquifer

3. Local Carrizo Program - Carrizo Aquifer

The proposed BGD takes advantage of existing plentiful but unused brackish groundwater from the Wilcox Aquifer. The brackish water will be treated by reverse osmosis and ultimately blended with treated water from the Local Carrizo and recovered ASR water.

The Local Carrizo Program was completed in 2010, and in this program raw water from the Carrizo Aquifer is transmitted to the Twin Oaks/ASR Facility where it is treated and sent to the distribution system via the ASR Pipeline.

The proposed Expanded Carrizo Program will drill additional wells in the Carrizo Aquifer and transmit additional raw water to the Twin Oaks/ASR Facility for treatment.

There will be a total of four water sources from three different aquifers that will all be treated and transmitted from the Twin Oaks/ASR Facility. With these additional sources, it has become clear that in order to utilize the ASR to its full potential and still provide water from the BGD and Local Carrizo Programs, another transmission pipeline is necessary. Because high growth and limited water availability is expected for the northwest area of San Antonio, the new pipeline has been planned to transmit water to the Anderson Pump Station.

COMPONENTS

The WRIP will be a 45-mile long 48” to 60” diameter steel pipeline which will generally follow a north-northwest alignment from the Twin Oaks/ASR Facility to the Anderson Pump Station (Anderson PS) as shown in **Figure 1**. Potable water will be pumped from the Twin Oaks West Pump Station to the Old Pearsall Road Pump Station where water will be integrated into SAWS’ Pressure Zone 4. Water will then be pumped to the existing Anderson PS, near the intersection of Loop 1604 & Hwy 151, where it will be integrated into SAWS’ Pressure Zones 7, 8, 11 and 12. Additionally, the WRIP has been designed for bidirectional flow. This will allow Edwards water to move from the Anderson and Old Pearsall Road Pump Stations to the ASR/Twin Oaks Facility where it can be injected into the ASR.

The WRIP is a program with four design components – three pipeline segments and the pump stations. The following is a list of each component:

- Pipeline Segment 1 Project
- Pipeline Segment 2 Project
- Pipeline Segment 3 Project
- Pump Stations Project

Each component is shown in **Figure 1** and described further in the next section.

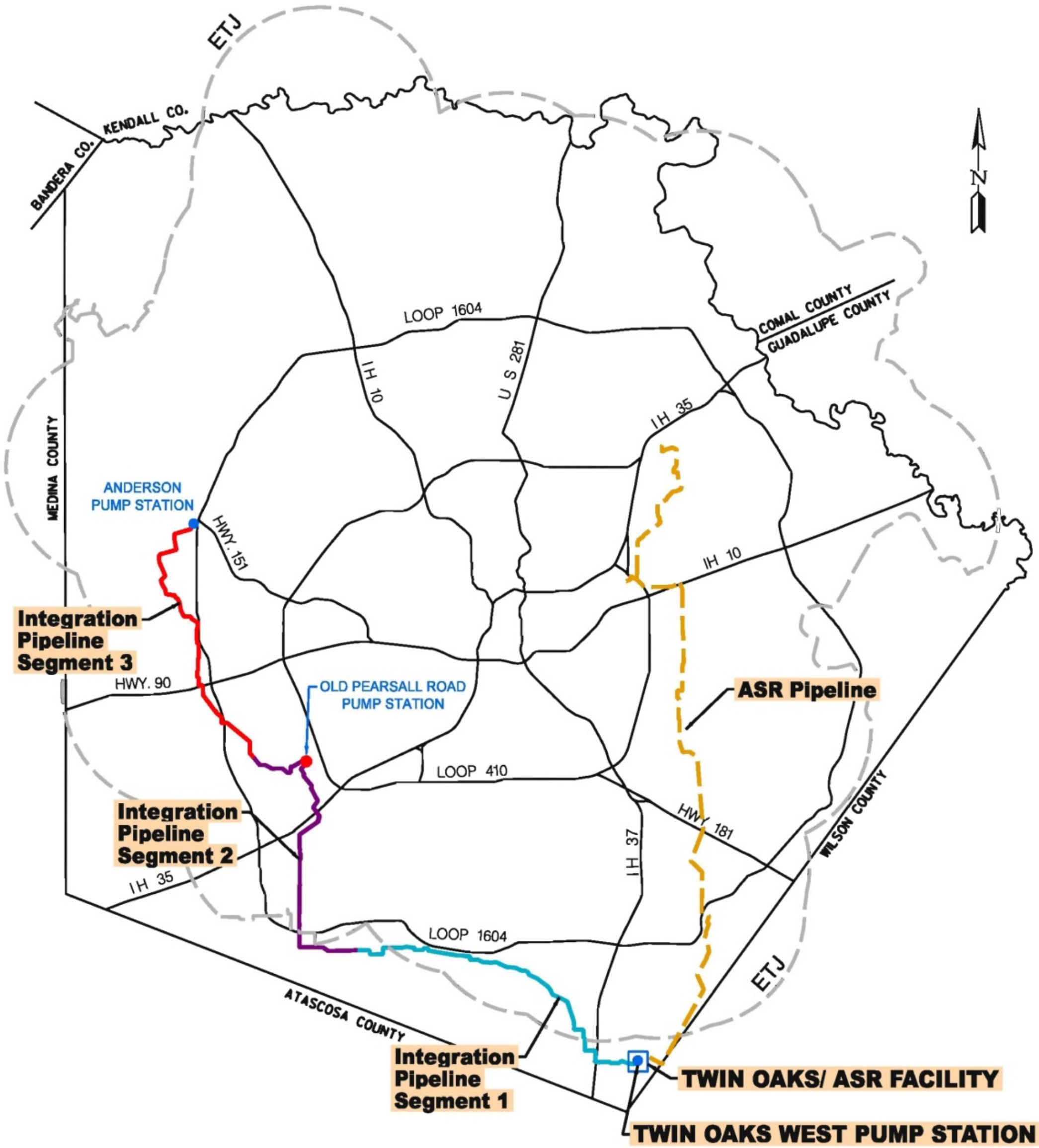

Figure 1. WRIP Layout

Pipeline Segments

Each of the three pipeline segments and the pump stations were designed by different consultants for a total of four different design packages.

The Southernmost portion is Segment 1 which is about 16 miles long and shown in **Figure 1** as a blue line. The pipeline starts at the Twin Oaks West Pump Station and extends in a western alignment through rural and agricultural areas in southern Bexar County.

Segment 2 is the middle segment and connects Segments 1 and 3. **Figure 1** shows it as a purple line. This segment is also about 16 miles long. This portion also crosses through predominantly rural and agricultural areas and includes the Old

Pearsall Pump Station. Design for portion of the pipeline to the west of the Old Pearsall Pump Station is currently on-hold at 90% completion, but design for the portion of the pipeline south of the pump station has already been completed.

The northern portion of the WRIP is Segment 3 and is shown as a red line in **Figure 1**. This segment is approximately 14 miles long and connects Segment 2 to the Anderson PS. Design for this segment is also on-hold at 90% completion. Similar to the other two segments; it generally crosses through rural areas south of Hwy 90, but north of Hwy 90 is an area of recent residential and commercial development.

Pump Stations

The WRIP requires two pump stations to move water from the Twin Oaks/ASR Facility to the Anderson PS. Both pump stations are shown in **Figure 1** and further described below:

- Twin Oaks West Pump Station - located at ASR Facility
- Old Pearsall Road Pump Station - located at the midpoint of the project, near the intersection of 410 and Old Pearsall Road.

The Twin Oaks West Pump Station will take treated water from the four sources and pump it to the Old Pearsall PS. At that location, the water will be split into one of two 7.5 million gallon (MG) ground storage tanks (GSTs). Water from the Old Pearsall PS GSTs will then be either pumped through the Old Pearsall Road Pressure Zone 4 Pumps (PZ 4 Pumps) and distributed to SAWS' customers in Pressure Zone 4, or pumped through the Old Pearsall Road Booster Pumps to the Anderson PS.

At SAWS' Anderson PS, the flow will be split between two existing 7.5 MG GSTs where it will be blended with Edwards Aquifer water for distribution to SAWS' consumers in Pressure Zones 4, 7, 8 11 and 12.

CONSTRUCTION PHASING AND FUTURE FLEXIBILITY

The intent of the WRIP is to provide flexibility, and many design features were incorporated to provide that flexibility. In order to discuss these features, however, there must be a general discussion of construction phasing.

Construction Phasing

In general, construction of the WRIP will be phased to coordinate with phasing of the BGD. Construction of the WRIP will be broken up into Phases I and II as shown in **Figure 2**. Because the WRIP is a large project, phasing construction will minimize the impact to the capital improvement project budget and allow for coordination of construction timing with the completion of the BGD Program. Construction of the BGD and WRIP will be broken up into two phases with increasing capacity as shown in **Table 1**.

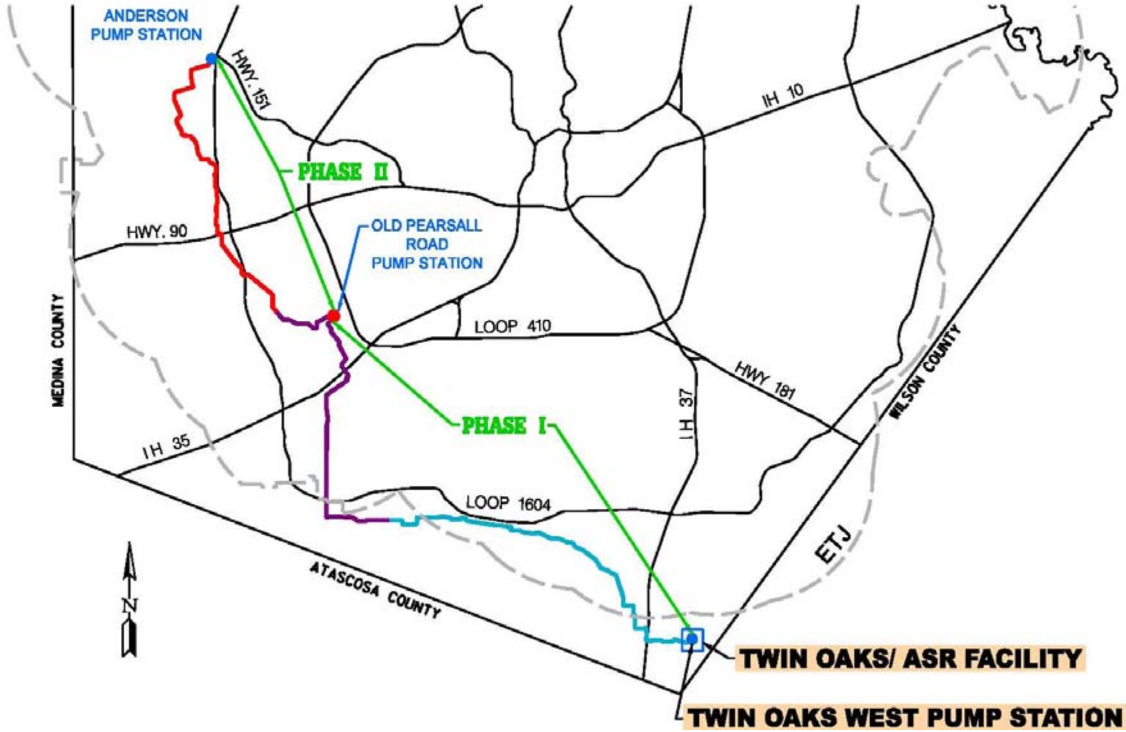

Figure 2. WRIP Construction Phasing

Table 1. Capacities of Phased Construction

Construction	Year Complete	Cumulative Capacity (MGD)	
		WRIP	BGD
Phase I	2016	45	10
Phase II	2021	75	up to 25 by 2026

*WRIP will also convey water from the Local and Expanded Carrizo Programs and recovered ASR water

Phase I construction of both the WRIP and BGD will be complete by 2016. Once Phase I is complete, the WRIP will have the capacity to move up to 45 MGD of water approximately 28 miles from the Twin Oaks PS to the Old Pearsall Road Pump Station. As mentioned, the WRIP will also have the capability to move water from the Old Pearsall Road PS to the Twin Oaks/ASR Facility, where water can be injected into the ASR. Construction of Phase I will only include booster pumps (equipped with variable frequency drives) that are necessary to move approximately 5 to 45 MGD of potable water from the ASR Facility to the Old Pearsall PS. Phase I construction of the Old Pearsall PS will only include one 7.5 MG GST, pumps to integrate water into SAWS' distribution system, and an electrical building. Construction for Phase I is estimated to be $99M and was divided into six construction packages which will start in 2015. Construction was split into six packages so the packages could be constructed simultaneously and help facilitate a faster schedule.

Phase II, the design of which is currently on hold, will include the remaining pipe, most likely 48” diameter, from Old Pearsall PS to the Anderson PS. Phase II will also include the installation of high-service booster pumps at the Old Pearsall PS to boost the remaining water approximately 17 miles to the Anderson PS. Construction of the Old Pearsall PS at that time will also include the second 7.5 MG GST. Because Phase II will move up to 75 MGD of water from the Twin Oaks/ASR Facility to Old Pearsall PS, two more 15 MGD pumps will be installed at the Twin Oaks PS. Again, this portion of the line will allow bidirectional flow, so water can be conveyed from the Old Pearsall PS to the Anderson PS, or the flow from Anderson PS to the Old Pearsall PS, and then to the ASR Facility where it can be injected into the ASR. Construction contracts for Phase II of the WRIP are scheduled to be awarded in 2020 and will likely be split into four construction packages to help facilitate a quicker construction schedule.

OPERATIONAL FLEXIBILITY

The WRIP mirrors the alignment as well as the design philosophy of the ASR Pipeline. When these two lines are used together, SAWS will have the operational flexibility to address a wide range of demand scenarios.

The benefits of the WRIP can be summarized as follows:

- Second pipeline to carry water from BGD, Local Carrizo, Expanded Carrizo and ASR
- Provide potable water to high growth areas
- Ability to integrate water from four different sources
- Will allow full utilization of the Twin Oaks Facility

These are further explained below in more detail.

Second Pipeline to Carry Water from BGD

The existing ASR Pipeline is currently the only pipeline between the Twin Oaks/ASR Facility and the SAWS distribution system. It is used to deliver water for storage at the ASR and then move water from the Local Carrizo and ASR in the distribution system. This arrangement limits production of the Local Carrizo to times when SAWS is not recharging the ASR. Once the BGD is operational, it will be very expensive to take any part of the system offline due to additional facility costs that would be incurred to maintain membranes taken out of service. The BGD, therefore, was designed to operate with a continuous base flow.

Provide Potable Water to High Growth Areas

In production mode, the WRIP will have the capacity to provide up to 75 MGD of potable water to west San Antonio – an area of high growth. Future demands in this area have made integrating additional water sources to western San Antonio critical to prepare for the drought of record. With water being integrated at the Old Pearsall Road and Anderson Pump Stations, the pipeline will provide water to Pressure Zones 4, 7, 8 11 and 12.

Ability to Integrate Four Different Sources of Water

The WRIP will integrate four different water sources from three different aquifers, to meet future demands. This is consistent with SAWS' goal of lessening San Antonio's reliance on the Edwards Aquifer. The WRIP, therefore, gives SAWS the ability to integrate water from the BGD, Local Carrizo, Expanded Carrizo and ASR, allowing reduced pumping from the Edwards Aquifer.

Will Allow Full Utilization of the Twin Oaks/ASR Facility

The WRIP will allow the Twin Oaks/ASR Facility to be utilized over a much wider range of demand scenarios which will be beneficial during a drought. For example, if Stage III water restrictions are in place, and irrigation is only allowed every other week, it would be very difficult to integrate 40 MGD through the ASR Pipeline during irrigation off-weeks. Construction of the WRIP is necessary for utilization of the Twin Oaks/ASR Facility during those times the facility is needed most.

SUMMARY

SAWS has the ability to store excess Edwards Aquifer drinking water in the Twin Oaks Aquifer Storage and Recovery (ASR) Facility during rainy seasons and recover it during dry periods. With the addition of three new water sources at the Twin Oaks/ASR Facility, however, an additional integration line is needed to allow SAWS to fully utilize these sources. Recovered ASR water and Local Carrizo water is already available, and with the completion of the BGD and Expanded Carrizo Program, an additional pipeline that mirrors the existing ASR Pipeline is even more critical. The WRIP, which will convey a total of 75 MGD of water once complete, will provide that solution. WRIP construction will be broken up into two phases with Phase I complete in 2016 for a total estimated cost of approximately $99M. Phase II will be online in 2021. The WRIP is being designed to provide ultimate flexibility and when used in conjunction with the ASR Pipeline, SAWS will be able to meet a wide range of supply and demand scenarios for high growth areas of San Antonio that are most in need of additional water sources.

Interconnections of the Lakeview Pipeline and Inland Feeder from Concept to Operation in 10 Months

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Abstract

The Henry J. Mills Water Treatment Plant (Mills plant) is one of five water treatment plants owned and operated by The Metropolitan Water District of Southern California. Located in Riverside, California, the Mills plantreceives virtually all of its raw water from the East Branch of the State Water Project (SWP), either by gravity from the Department of Water Resources' (DWR's) Santa Ana Valley Pipeline or via pumping from Lake Perris, another DWR facility. Because of the current state-wide drought in California, and resulting low allocation of SWP supplies, Metropolitan recognized that sufficient quantities of SWP water may not be available to meet demands at the Mills plant and that another source of supply was needed. Metropolitan staff reviewed several options and ultimately proposed using Metropolitan's Diamond Valley Lake (DVL) as a source for the Mills plant, even though at the time, Metropolitan's infrastructure did not allow the Mills plant to access the water in DVL. Metropolitan's solution was to modify the existing infrastructure, including a pressure control facility, a pumping facility, a tunnel, and two pipelines, so that water stored at DVL could be supplied to Mills Plant. This paper will describe the required modifications, discuss the design and construction of the interconnection, and address the methods used to expedite the project delivery so that a project that would usually take 30 months to complete was completed in just 10 months.

INTRODUCTION

The Metropolitan Water District of Southern California was created in 1928 to construct and operate the Colorado River Aqueduct (CRA). In addition to the 242-mile-long CRA, Metropolitan's system currently includes five regional water treatment plants, nine reservoirs, 16 hydroelectric plants, 830 miles of large-diameter pipelines and tunnels, and approximately 400 connections to member agencies. Metropolitan delivers, on average, about 1.7 billion gallons of treated and untreated water per day to the roughly 18 million people living within its 5,200-square-mile service area (see Figure 1).

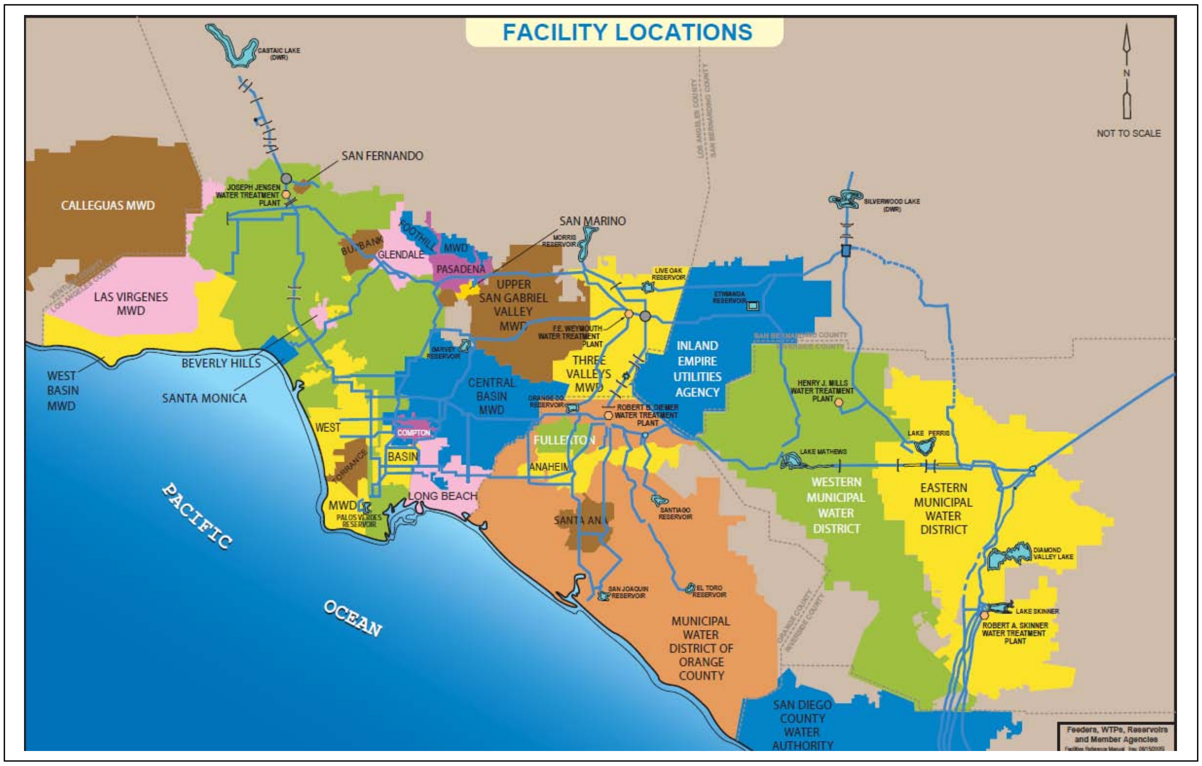

Figure 1: Metropolitan's Service Area

A Planned Interconnection Becomes an Urgent Connection

As part of its long-term planning, Metropolitan had intended to include an interconnection between its Lakeview Pipeline and Inland Feeder as part of a larger rehabilitation of the Lakeview Pipeline. The original plan was to connect Lakeview and Inland Feeder at the existing PC-1 pressure control structure (see Figure 2). However, as a result of drought conditions, Metropolitan re-prioritized this work and proceeded with the interconnection prior to the rehabilitation work on Lakeview Pipeline.

Planned Rehabilitation of Lakeview Pipeline and Bernasconi Tunnel

The planned rehabilitation of the Lakeview Pipeline and Bernasconi Tunnel involved installing a steel liner in both the pipeline and the tunnel. The rehabilitation was required to address ongoing problems with the pipeline and to provide additional reinforcing for both the pipeline and the tunnel.

The Lakeview Pipeline is an 11-foot-diameter pipeline with bell and spigot joints that is approximately 11.5 miles long. Since the pipeline was built in 1972, it has had significant problems, and addressing these problems has required that the pipeline be shut down on numerous occasions. With regard to the leakage issue, for example, 139 of the pipeline's 1,520 joints have required remedial repairs. The deflection issues were apparent even during construction and were measured in 2012, when staff conducted an internal 3D survey of the pipeline. In that survey, staff measured pipe deflections in excess of 4 inches at over 660 locations (the locations were scattered throughout the entire pipeline) with deflection of 12 inches at four locations.

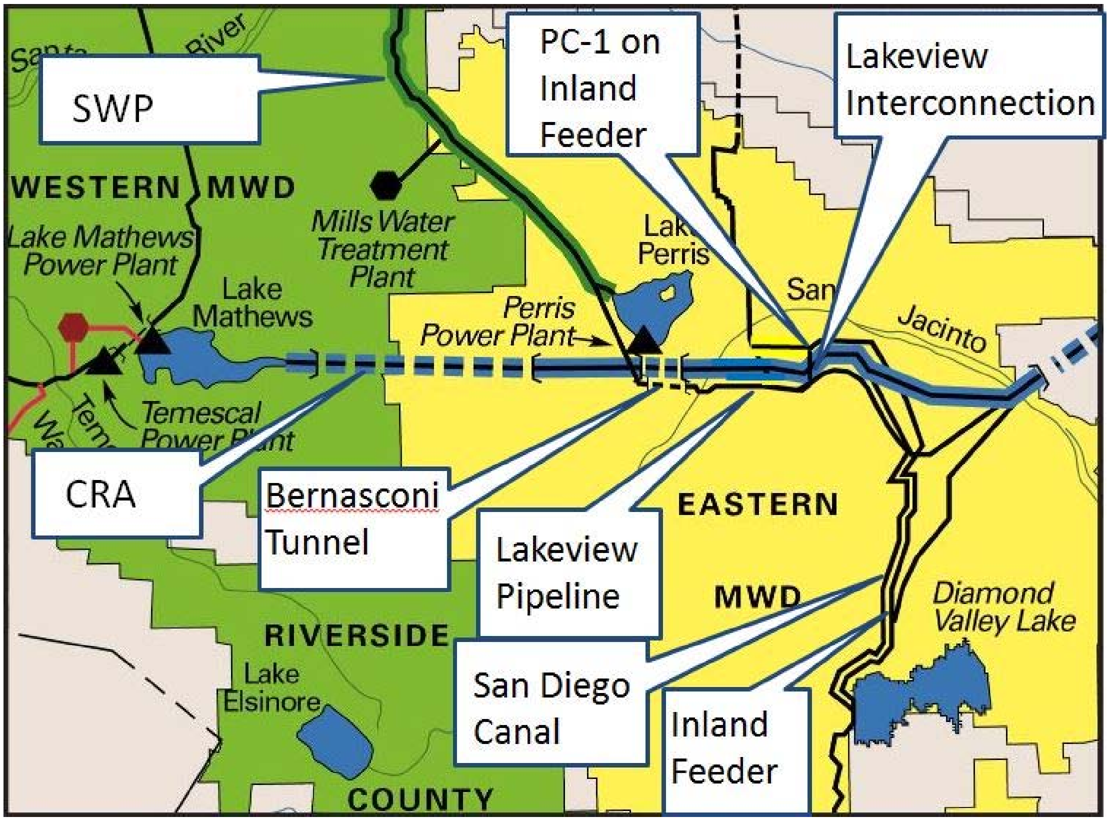

Figure 2: Lakeview- Inland Feeder Intertie

In view of these problems, Metropolitan determined that the pipeline needed a new steel liner. The addition of a new steel liner would address the problem with leaking joints and deflection, with the added benefit of allowing the pipeline to accommodate a higher pressure. The new steel liner would be designed to resist the maximum possible hydraulic grade line of 1,937 feet, which was significantly higher than the original HGL of 1440 feet.

Once Lakeview was lined, there would be an opportunity to provide increased reliability to the Mills plant. This would be accomplished by interconnecting the Inland Feeder and the Lakeview Pipeline at PC-1 (see Figure 3). This interconnection would be able to provide up to 340 cfs by gravity to the Mills Plant (which is greater than its current capacity of 248 cfs) from DVL via the Inland Feeder and Lakeview Pipeline.

The Bernasconi Tunnel is a one mile long, 11 diameter concrete-lined tunnel with minimal reinforcing at the portals and no reinforcing in the main body of the tunnel. This tunnel was originally designed to resist an HGL of 1440 feet (about 50 feet of pressure). The Lakeview Pipeline connects to the Bernasconi tunnel near Lake Perris. The rehabilitation of Lakeview Pipeline required that the Bernasconi Tunnel be lined with steel pipe to resist the higher head.

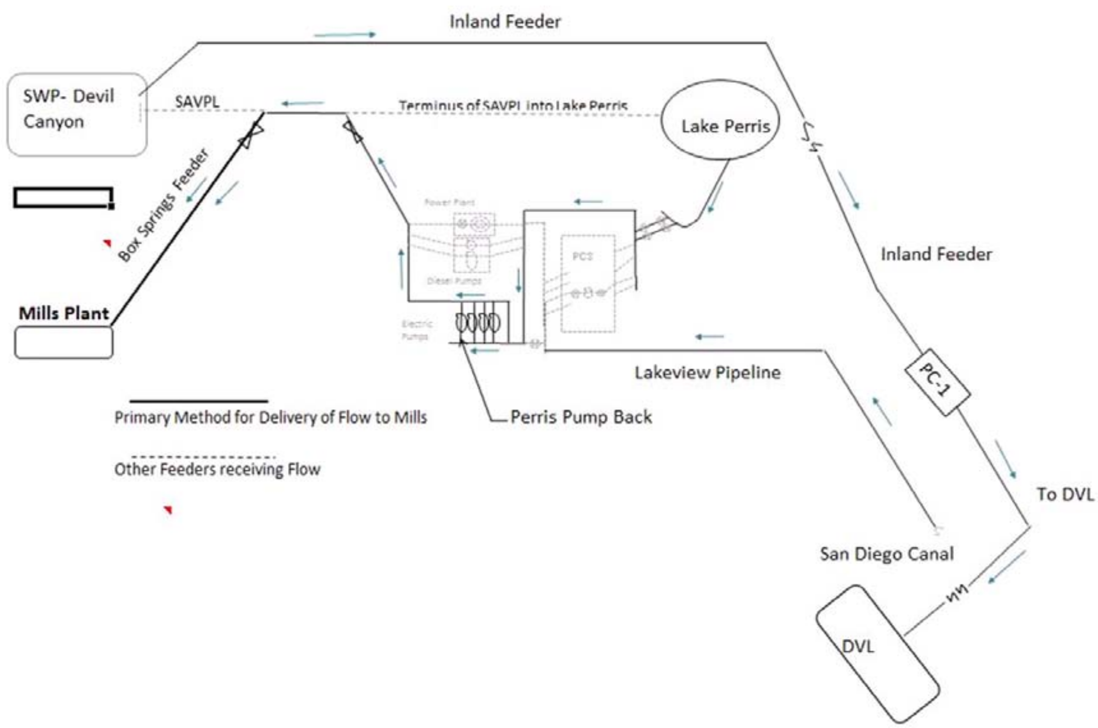

Figure 3 - Existing System, Flow from Lake Perris to Mills Plant

Existing Facilities – Lake Perris and the Perris Pump Back

Lake Perris is a reservoir on the SWP. It is downstream of the Mills plant and can supply water to the Mills Plant via the Lake Perris Pump Back facility. The Lake Perris Pumpback facility consists of four electric pumps with a nominal capacity of 40 CFS each. The piping has the flexibility to pump directly from Lake Perris or from the Lakeview Pipeline and deliver this flow to the Mills plant.

Metropolitan recognized that this pump station could be used to pump the water in DVL to Mills via the Perris pumpback facility after the interconnection if there was inadequate pressure to deliver the flow by gravity (Figure 4). If the water in DVL could be delivered by gravity, then the Perris pump back facility had to be bypassed (Figure 5).

Existing Facilities –PC-1 Control Structure

The Inland Feeder Pressure Control Structure (PC-1) controls the flow of water through the Inland Feeder to its target destinations. It regulates flow and pressure in the Inland Feeder and can be configured to deliver flow from the Inland Feeder to DVL or to the CRA.

PC-1 controls the flow of SWP to DVL or the CRA by using six motor-operated 54-inch vertical sleeve valves. Flow though the six sleeve valves discharges into a large steel outlet pressure chamber. Each sleeve valve controls the rate of discharge into the pressure chamber and safely reduces the pressure of the water. The water in each chamber can then be directed to either the CRA (HGL 1400) or DVL (HGL 1784).

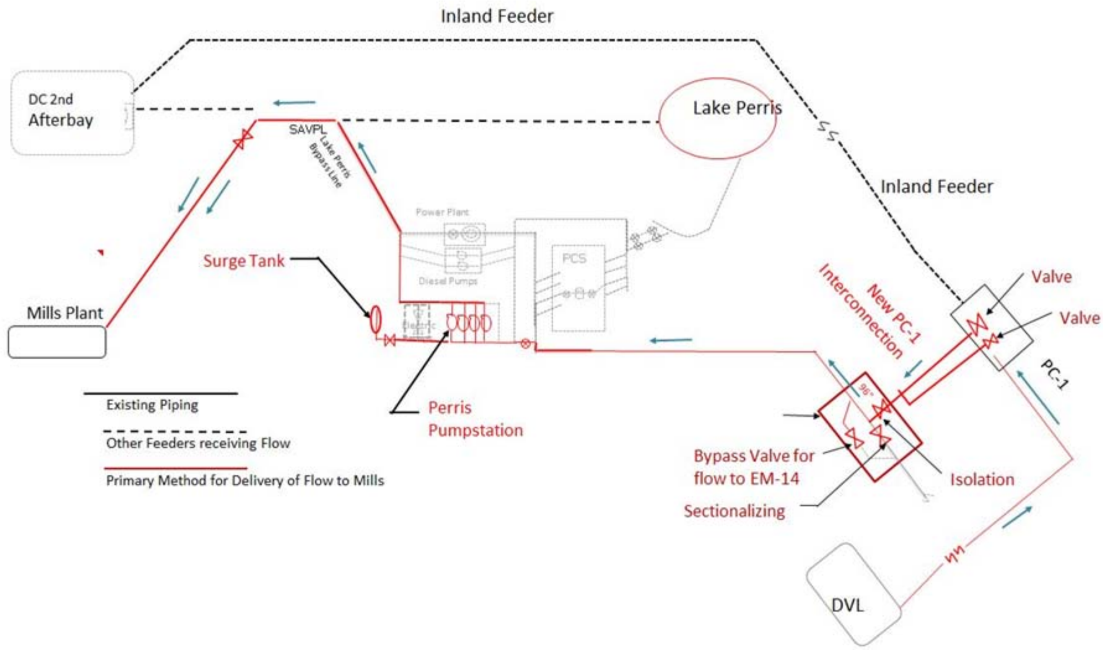

Figure 4–DVL to Mills Plant (pumping at Perris)

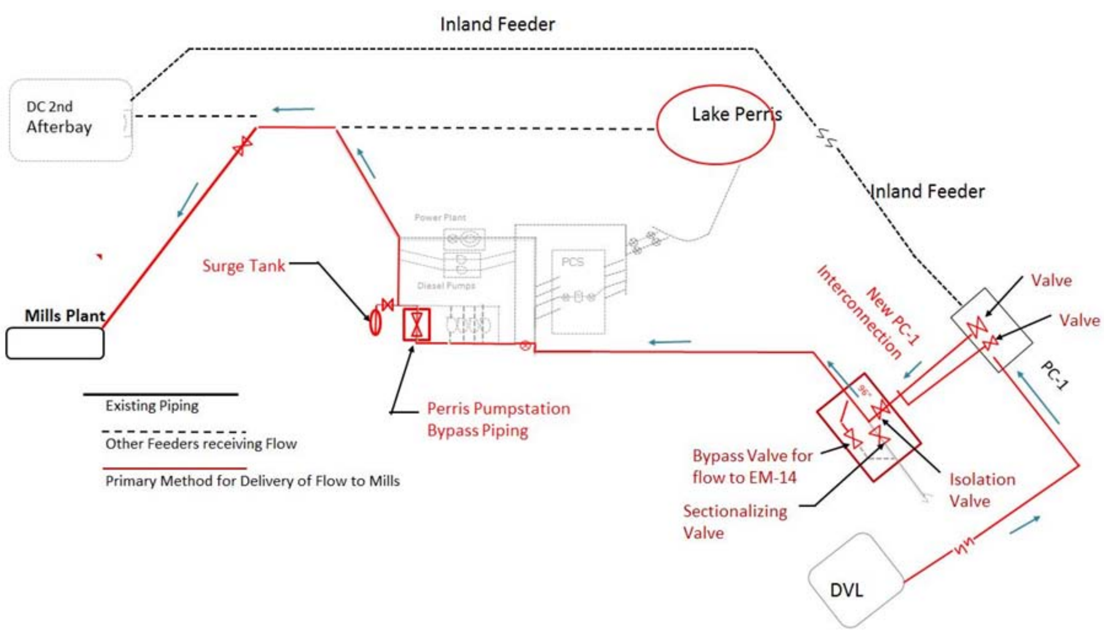

Figure 5–DVL to Mills Plant (gravity flow)

Two of these sleeve valves were originally constructed with discharge pipes that were intended to be connected to the Lakeview Pipeline. These discharge pipelines were included in the original construction and bulk-headed off because MWD recognized that PC-1 could eventually be interconnected to Lakeview Pipeline and could be sued to control the pressure in the Lakeview Pipeline and control the flow to Mills.

Urgent Need – California Drought

California is currently in the third year of a severe drought. In 2014, the state received the second lowest snowfall in the last 100 years. As a result of the drought, the SWP reduced its planned allocation to its member agencies to only 10% of the requested allocation (which was a record low allocation). Since the Mills plant can only receive water from the SWP, MWD was concerned that there may not be adequate state water available to supply to the Mills plant. MWD decided to accelerate the planned interconnection between the Inland Feeder and the Lakeview Pipeline so that water stored at DVL could be delivered to the Mills Plant via the Lakeview Pipeline. The PC-1 pressure control structure would be used control the flow and pressure in the Lakeview pipeline as required to deliver water safely to the Perris Pump Station.

The project

In January 2014, MWD began the interconnection project. The connection was to be completed and the system to be on-line as fast as possible. The schedule called for construction to be complete by October 30, 2014, or just ten months after the project began.

The project team realized that the Lakeview pipeline could not be lined in the time frame required. To line the Lakeview pipeline as originally planned would require almost 4.3 miles of pipe to be lined (i.e. the Lakeview Pipeline between PC-1 and Lake Perris). So it was determined to limit the head in the existing pipeline to the pressure the existing pipeline could withstand.

The first step in the design process was to determine the maximum allowable pressure the Lakeview Pipeline could resist. It was determined that the pipeline could resist the head in DVL (HGL = 1784 ft), provided the thrust in the bends was anchored. Fortunately, there was only one significant bend. The design team decided that this bend would utilize internal buttstraps to anchor its thrust and not just weld the existing bell to the existing spigot.

The Bernasconi tunnel, however, would not be able to take the DVL head of 1,784 ft. In fact, Bernasconi Tunnel could only take the original design pressure of 50 feet of head (HGL= 1440 feet), with no surge allowance. The reason the allowable pressure was so low was that the tunnel had only minimal reinforcing at the portals and no reinforcing in the center of the tunnel.

The hydraulic limits of the Bernasconi Tunnel meant that the DVL pressure had to be reduced at the PC-1 pressure control structure so that the maximum design pressure at the Bernasconi tunnel did not exceed the original design. It also meant that the Perris Pump Back facility would be required to pump the water to Mills Plant instead of using gravity flow. The project team recognized that management may well line the Bernasconi Tunnel in another contract before the Lakeview pipe lining was completed. If the tunnel was lined then water could be delivered by gravity to Mills Plant, which was the preferred method of operation. So the team decided to design this project for both a lined Bernasconi Tunnel and an unlined Bernasconi Tunnel (See Figure 4 and 5).

PC-1 Interconnection

The next design feature was to connect PC-1 to the Lakeview Pipeline. There were two existing 54 inch diameter pipelines (stub outs) at PC-1 which would be used as connection points (one at each sleeve valve designed to provide flow to the Lakeview Pipeline). These connection pipelines were 15.5 feet below grade and immediately adjacent to the PC-1 structure. The two 54 inch lines needed 60 inch isolation valves, so the lines were upsized to 60 inches. The two lines then merge to become one 96 inch line, which then continues and ultimately connects to the Lakeview pipeline.

Along the alignment of this 96 inch line, there were two large pipelines running perpendicular to the alignment that had to be crossed. The first was the 144 inch Inland Feeder, which was about 75 feet from PC-1, and the second was the CRA, which was about 60 feet from the Lakeview Pipeline. The CRA is a cut and cover conduit with about 4 feet of cover. The CRA is not designed for traffic loading but can carry a significant dead load. The design team realized that crossing under the CRA was not realistic because the time required to excavate a tunnel under the CRA would not permit the interconnection to be completed in time. So the design called for open excavation and placing the interconnection piping above the CRA. To facilitate this, the cover over the area was increased so that the new 96 inch line could be buried instead of only being partially buried. An analysis was performed to insure the CRA would not settle differentially.

The design of the interconnection required several design features. First, the downstream portion of the Lakeview Pipeline had to be isolated from the upstream portion of the Lakeview Pipeline. This would force flow from PC-1 to Mills Plant. Second, the connection required the ability to isolate the interconnection pipeline from the Lakeview Pipeline if the Lakeview Pipeline was being used to convey water to the downstream service connection EM-14 (see Figure 4). Third, it was required that the downstream service connection EM-14 be able to get flow from PC-1 whether sending flow to Mills Plant or not. In addition, the head on the downstream portion of the Lakeview Pipeline had to be controlled so that the Lakeview Pipeline would not be over pressurized.

In order to meet these requirements, two 84 inch valves and a 24 inch throttling valve were required. Metropolitan fortunately had two 84 inch butterfly valves available which were rated for the pressure. One 84 inch valve would be used to isolate the Lakeview Pipeline from the PC-1 interconnection, and the other 84 inch valve was installed on the Lakeview Pipeline and would prevent flow from the interconnection piping from going downstream (see Figure 4). The 24 inch throttling valve was plumbed so that the valve could deliver water from PC-1 to downstream of the 84 inch valve or could deliver water from upstream the side of the Lakeview isolation valve.

Another element of this interconnection was an 84 inch by 84 inch wye connection with 4 inch thick crotch plates. Since the valves were only 84 inches, the 132 inch Lakeview pipeline needed two eccentric reducers to accommodate the valve. The interconnection had two vaults to house the two valves.

Perris Pump Back Facilities

The Perris Pump Back facilities also required modifications. For one thing, there was no way to bypass the pumps to allow gravity flow. A full size bypass pipeline around the pumps was proposed that would enable flow to get to the Mills Plant by gravity when the Bernasconi Tunnel was lined. The suction line to and from the pumps was 60 inches, so a 60 inch bypass pipeline was installed adjacent to the pumps. This bypass line required two 60 inch by 60 inch tees with 1.5 thick inch crotch plates. A 60 inch valve was used to isolate the line if the pumps were required.

The second feature required at Perris was a surge tank. If the Bernasconi Tunnel was unlined, the allowable pressure plus surge in the Bernasconi Tunnel had to be limited so that it could not exceed the original static pressure. This meant that the Perris Pump back was required to pump the flow to Mills Plant. However, if the pumps were to trip, the surge pressure in suction piping (the Bernasconi Tunnel) would exceed the allowable pressure. Calculations showed that a 12 foot diameter by 50 foot long surge tank was required to prevent over pressurization of the Bernasconi Tunnel.

The design called for the ability to connect the surge tank to the discharge side of the pumps once the Bernasconi Tunnel was lined. That is because a surge tank had been recommended for the discharge piping but was never installed due to the infrequency of pumping and the ability to limit the surge pressure by the use of special operating conditions. However, with a surge tank available after the Bernasconi Tunnel was lined it was preferred to use it so that the special operating conditions would no longer be required. So piping was installed to allow the surge tank to be on either the suction or discharge side of the pumps (see Figures 4 and 5).

The schedule

The schedule was extremely aggressive: The design would be completed in just 14 weeks, the advertise period would be 5 weeks, and after bid opening, the Board would award the contract in just 10 days. The notice to proceed would be given in just 5 days and construction of the portions of the project required to allow flow was to be completed in just 20 weeks.

In order to complete the design in 14 weeks, the overall strategy was simple: have a dedicated project team that will work exclusively on this project and take whatever steps are required to meet the project schedule. First, the project manager took care of project permits, environmental, hydraulic or other operations requirements, and insured that all project needs were being met. Simultaneously, the design manger focused on completing the design. To do that, there were weekly progress meetings and other meetings as required to keep the project moving. The design manager also kept in touch with each discipline on a weekly basis, making sure each discipline had the required information. When resources became an issue (when it was determined that a surge tank was needed, for example, with only about 3 weeks remaining in the design schedule), a consultant was brought on board and they completed the design. The design team used a consultant familiar with Metropolitan standards and had a track record of success completing fast track projects.

An important part of the strategy was to use our best mechanical, structural and civil designers to layout the facilities and determine the pipeline alignment. This insured that the facilities had less rework because they were well thought-out initially. Once the layout was complete, Water System Operations (WSO) personnel responsible for the facility were consulted to insure the on the layout of vaults met their needs and that there were no fatal flaws in the layout. Once the layout was agreed upon, other staff was assigned to complete the design and determine the specific sizes of walls, rebar, pipe thicknesses and grading details.

Another part of the strategy was to have electrical designers and engineers start almost immediately. This insured that any electrical concerns were addressed early and maximized the design time for the electrical discipline. This enabled the electrical engineer and designer to finish on schedule.

One of the most important strategies for completing the design so quickly was to use a separate engineer for every major component. The structural engineers had one engineer responsible for all piping and fabricated fittings, and two engineers were responsible for the design of vaults. The civil engineers were broken into two parts, one for Perris Pump Back and one for the Lakeview intertie. There were two mechanical engineers (one for valve procurement and one for design of the vaults). A consultant was responsible for the surge tank design, including the required electrical drawings. The Design Manager and Project Manager put together all of the shutdown requirements and other scoping documents for the specification.

The final strategy to complete the design in 14 weeks was to have only two reviews. One review occurred about 50% of the way through the design effort and the other when it was effectively complete. The design reviews were only one week long, with each review period consisted of a two day review period followed by a review meeting. All comments were resolved at the review meeting.

Due to the time constraints and shutdown related work on this project, only prequalified bidders were allowed to bid. This insured that only highly competent Contractors would be bidding the job. When the bids were opened, JF Shea was the low bidder, with a lowest responsive bid of $20,365,430.

The award process was also expedited. Normally, a project is not awarded by the board in less than 5 weeks after bid opening. For this project, however, Board approval was achieved in just 7 days. In order to facilitate such a short period between bid opening and award by the Board, Engineering and WSO management explained the situation to the GM and Board. The GM and Board recognized the special circumstances, and this allowed for an accelerated board action.

To complete the construction, all submittals had only a 5-day maximum turnaround. Many of required submittals were provided to Metropolitan by the contractor in the first 20 days. The design team had to prioritize these submittals above all other work. Anyone on leave had to have someone else designated to review his or her submittals. The contractor also indicated the priority of the submittals, since some submittals could take more than 5 days and others needed to be turned around in less than 5 days, if possible. Examples of the expedited nature of this project include:

- The pipe shop drawings were approved on June 16 and the first pipe was delivered to the site on July 10.
- The surge tank shop drawing was approved on August 21 and arrived on the site on September 30.
- The shoring for the vaults was received on June 24 and excavation began on June 26.
- The rebar for the vaults was submitted on July 21 and the contractor began placing rebar on July 23.

Another key to completing the construction on time was recognizing that not everything had to be completed. Only the components that were required to deliver water were required. That meant all piping, valves, and their supports had to be completed but the vault's roof and platforms did not have to be completed. The design package spelled out exactly what had to be completed in order to deliver water.

One of the biggest concerns for the schedule was the delivery of the valves. Most valves over 30 inches have a very long lead time. As mentioned earlier, Metropolitan was fortunate in that two 84 inch valves from a previous project that had been canceled were available. The 24 inch throttling valve was also available because it had recently been rehabilitated after being removed from another location. The three 60 inch valves required for the project were ordered, but due to the long delivery time, the valves were not installed and a spool piece was installed instead. All of the small valves were ordered by Metropolitan during the design process and provided to the contractor.

Conclusion

The contractor was given notice to proceed on June 19, 2014 and construction began almost immediately. On October 30, 2014, just 4.5 months after NTP, the intertie was ready to deliver water. In that short period, the contractor manufactured and laid 450 feet of pipe (sizes varied from 54-inch to 96 inch diameter) that included two 60 x 60 inch tees with crotch plates and a 84 x84 inch wye with 4.5 inch thick crotch plate; constructed portions of four valve vaults, including the floors and walls; installed the piping and valves; tested the valves; installed the required electrical and instrumentation; installed a surge tank, compressor, electrical power and instrumentation and ancillary piping required for its operation; installed a 134 inch bulkhead at EM-14; and removed two bulkheads at PC-1 and two bulkheads at Perris Pump Back.

This project demonstrated that it is possible for a public agency to conceive and complete a significant project in short order. It takes dedication, clear direction from upper management, experienced design personnel, sound project management, dedicated and knowledgeable construction inspection and a good Contractor.

Proposed Simplified Changes to ANSI/AWWA C304 Standard for Design of Prestressed Concrete Cylinder Pipe

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Abstract

In 1992, the American Water Works Association (AWWA) introduced a new standard for the design of prestressed concrete cylinder pipe (PCCP) designated as ANSI/AWWA C304. The ANSI/AWWA C304 Standard introduced a new design philosophy for PCCP based on the concept of evaluating and satisfying certain serviceability, elastic and strength limit-states criteria using various combinations of factored and unfactored design loads and internal pressures. When the C304 Standard was first introduced in 1992, many potential users voiced concerns related to the complexity of the design provisions contained therein that continue to date. Upon performing a detailed review of the C304 Standard, it is apparent to the authors that some of the provisions currently incorporated in the document can be simplified without any significant corresponding consequences relative to the final overall design solution. This paper addresses some of these overly-complex design provisions and provides alternate or simplified provisions for a few of them. In the near future, the authors anticipate performing a more comprehensive evaluation and preparing additional supplementary publications with the intention of providing further justification for simplifying future versions of the ANSI/AWWA C304 Standard.

INTRODUCTION

In 1992, the American Water Works Association (AWWA) introduced a new standard for the design of prestressed concrete cylinder pipe (PCCP) designated as ANSI/AWWA C304. Prior to introduction of the C304 Standard, PCCP had traditionally been designed using either the Cubic Parabola Design Method or the Stress Analysis Design Method described in Appendix A or Appendix B of the ANSI/AWWA C301 Standard, respectively. While it can be argued that earlier design methodologies left much to be desired from a technical standpoint, they still provided satisfactory solutions as evidenced by the fact that performance shortfalls of pre-1992 vintage pipes have generally not been attributed to design issues.

Prior to the introduction of the ANSI/AWWA C304 Standard, the provisions that governed the design of PCCP were effectively communicated in just a few pages provided at the end of the C301 Standard (Appendices A and B). With the introduction of the C304 Standard, the design provisions for PCCP that were previously communicated in a very concise manner now required a document containing over 100 pages plus another 50 pages of appendices. While it may be argued that the C304 Standard provides a more complete and technically-correct approach to PCCP design than previous methods, most users would agree that the design method is difficult to understand and follow and is virtually unusable without reliance on proprietary software.

Due to the inherent complexity, the C304 design methodology essentially requires the use of a proprietary computer program (known as the Unified Design Program) that has been made available through the American Concrete Pressure Pipe Association (ACPPA). The software is offered free of charge to ACPPA members and at a relatively substantial cost of $5,000 for non-members. The third revision of the ANSI/AWWA C304 standard is scheduled to be released this year. However, no significant attempts have been made to date to modify or simplify the C304 design provisions that were introduced in the original 1992 version of the Standard.

When the ANSI/AWWA C304 Standard was first introduced, many potential users (including the authors) questioned the complexity of the design provisions contained therein. Having served on the AWWA Standards Committee on Concrete Pressure Pipe for over a decade, one of the authors can vouch for the fact that the balloting for each of the three revisions of the ANSI/AWWA C304 Standard (1999, 2007 and 2014) has been consistently met with opposition and has resulted in the discovery of errors and unresolved comments from reviewers.

It is apparent that some of the provisions currently incorporated in the ANSI/AWWA C304 Standard can be simplified without significant corresponding consequences relative to the final overall design solution. In addition to being overly-complex, there are other issues with the C304 Standard which can and should be resolved to enhance usability. Finding ways to simplifying the Standard constitutes the first logical step toward tackling some of the issues that currently exist. This paper addresses some of the C304 design provisions that are deemed overly-complex and provides alternate or simplified provisions for a few of them. In the near future, the authors anticipate performing a more comprehensive evaluation and preparing additional supplementary publications with the intention of providing further justification for simplifying future versions of the ANSI/AWWA C304 Standard.

ASPECTS OF C304 DESIGN METHOD WHERE SIMPLIFICATION MAY BE POSSIBLE

Our preliminary review of the ANSI/AWWA C304 Standard resulted in the identification of several aspects of the design method where simplification appears possible. The following are five fundamental questions that, once answered, might provide useful insights related to potential simplification:

- **Is it possible to simplify the fourteen pages of equations in Section 8.9 of the ANSI/AWWA Standard that relate to the computation of stresses and strains in the pipe wall for evaluating serviceability, elastic and strength limit states?** Evaluating serviceability, elastic and strength limit states involves calculating certain pipe material stresses, material strains, and pressures resulting from one or more applicable load combinations. The equations in Section 8.9 of the C304 Standard are used for this purpose. An initial examination of these equations indicates that several could be significantly simplified or eliminated altogether. As an example, the simplified form of some cumbersome and hard to understand equations of Section 8.9.1 of the Standard are presented later in this paper. These simplifications make the equations more understandable to the user.

- **Is it necessary to directly determine and utilize multipliers for the concrete modulus of elasticity (C_E), creep factor (C_ϕ) and shrinkage strain (C_s)?** Design material properties are presented in Section 5 of the ANSI/AWWA C304 Standard. The majority of the content of Section 5 consists of provisions to be followed to determine multipliers for the concrete modulus of elasticity (C_E), creep factor (C_ϕ) and shrinkage strain (C_s). Prior to the introduction of the C304 Standard, these multipliers were not deemed necessary or included in the design methodologies for PCCP. However, the provisions of the C304 Standard now include the requirement that PCCP manufacturers perform standard tests on molded cylindrical test specimens to facilitate evaluation of the multipliers. There is an abundance of available technical literature indicating that concrete properties such as modulus of elasticity, creep and shrinkage are influenced by certain fabrication, construction and in-service conditions. In a pipeline, perhaps more so than any other structure, conditions related to construction sequence, in-service exposure and operation can vary considerably. When all of these potential variables are considered, along with the circumstance that many of these variables cannot be reliably accounted for during the design stage, it can be concluded that attempts to optimize or fine tune certain aspects of design are not warranted given the uncertainties involved. Discussion related to each of the three aforementioned multipliers is provided later in this paper. A more comprehensive evaluation of this issue, along with specific recommendations for simplification, will be the subject of a future article.

- **Is it necessary to evaluate fourteen different load/pressure combinations and three different limit states criteria (i.e., serviceability, elastic and strength)?** Although the rationale and limiting criteria are communicated in the C304 Standard with reasonable clarity, it is unclear at this time whether it is necessary to evaluate all combinations and criteria. Manufacture of PCCP is governed by the ANSI/AWWA C301 Standard. Hence, the manufacturing and design parameters for PCCP are somewhat limited by certain provisions contained in the C301 Standard and the capabilities of the various pipe manufacturers. Recognizing that the range of design possibilities for PCCP are limited, performing parametric studies using the current C304 design provisions to evaluate the need for all the current load/pressure combinations and limit state criteria is feasible. Hence, it may be possible to eliminate some of the load/pressure combinations and limit states criteria if it can be shown through parametric studies that they will never have a controlling influence on the overall design solution. A more comprehensive evaluation of this issue, along with specific recommendations for simplification (where deemed appropriate), will be the subject of a future article.

- **Is it necessary and appropriate to attempt to account for the effects of environmental exposure conditions (outdoor or burial) and exposure duration in design?** As stated in the ANSI/AWWA C304 Standard, the method of calculating residual stresses in the concrete core, the steel cylinder, and the prestressing wire separately accounts for the effects of elastic deformation, creep, and shrinkage of concrete, and the relaxation of the prestressing wire. Although it is certainly necessary and appropriate to account for factors that affect the resultant or final prestress level in the concrete core, the detailed methodology described in the C304 Standard requires information that often cannot be reliably predicted or known during the pipe design stage. Pipe exposure intervals from time of manufacture to time of burial or from time of burial to time of filling with water are examples of such information. Hence, while it may seem appropriate and technically correct to attempt to account for environmental exposure when calculating certain pipe wall stress levels, it may be possible to demonstrate through parametric studies that this is not necessary and that the use of certain simplified assumptions will suffice. A more comprehensive evaluation of this issue, along with specific recommendations for simplification (where deemed appropriate), will be the subject of a future article.

- **Is it necessary to directly account for the weights of the pipe and pipe contents in the design procedure?** The C304 Standard currently includes these two working load effects (i.e., pipe weight W_p and fluid weight W_f) in all fourteen of the required load/pressure combinations identified in Sections 3.4 thru 3.6. The standard also establishes eight different moment and thrust coefficients to be utilized when evaluating pipe wall stresses and strains attributed to these effects (C_{m1p}, C_{m1f}, C_{m2p}, C_{m2f}, C_{n1p}, C_{n1f}, C_{n2p}, *and* C_{n2f}). Although the weight of the pipe and contents gradually becomes more significant with increasing pipe size, for any given design scenario, these parameters can conservatively be viewed as a constant source of working load. Hence, it stands to reason that it may be possible to account for these effects using an indirect approach that would ultimately reduce the overall design effort required. A more comprehensive evaluation of this issue, along with specific recommendations for simplification (where deemed appropriate), will be the subject of a future article.

The remaining portion of this paper is devoted to discussion of the first two bullet points that identify two aspects of the ANSI/AWWA C304 Standard where simplifications can clearly be made. The first aspect relates to the provisions contained in Section 8.9 of the C304 Standard that include equations for calculating pipe component material strains and stresses needed to evaluate limit states criteria. The second aspect relates to the multipliers for the concrete modulus of elasticity (C_E), creep factor (C_ϕ) and shrinkage strain (C_s) discussed in Section 5 of the ANSI/AWWA C304 Standard. Although not part of the evaluation work performed to date, the authors plan to undertake parametric studies in the near future using the current C304 design procedure to determine if any simplifying assumptions can be made relative to the last three fundamental questions posed above.

SIMPLIFICATION OF PROVISIONS RELATED TO EVALUATION OF LIMIT STATES CRITERIA

Section 8 of the AWWA C304 Standard incorporates provisions for evaluating serviceability, elastic and strength limit states. Specific criteria for evaluating these limit states are given in Tables 3 and 4 of the Standard for embedded-cylinder and lined-cylinder pipe, respectively. Evaluating these limit states involves calculating certain pipe material stresses, material strains, and pressures resulting from one or more applicable load combinations. These calculated stresses, strains and pressures are then compared to corresponding limiting criteria associated with a given serviceability, elastic and strength limit state.

One aspect of Chapter 8 where simplifications can clearly be made involves the provisions for calculation of pipe component material strains and stresses defined in Section 8.9. Section 8.9 consists of fourteen pages incorporating text, diagrams and equations used for computation of stresses and strains in the pipe wall resulting from bending moments and thrust forces acting at the crown, invert and springline. Upon detailed examination, it is apparent that several of these equations could be simplified, and some could be eliminated altogether.

Consider the computations of stresses and strains in the pipe wall subjected to moments and thrusts described in Section 8.9.1 for the invert and crown. The strain equations express the strains at the critical points of the pipe wall using the assumed value of strain at the inside surface of the core as expressed by the dimensionless factor ν_2 and the assumed strain gradient expressed by the dimensionless factor k. The stress equations are based on the assumed material stress–strain relationships for core concrete, mortar coating, steel cylinder and prestressing wire defined in Section 5 of the C304 Standard. The assumed strain and stress distributions at the invert and crown of an embedded-cylinder pipe are given in Figure 5 of the Standard and are shown in Figure 1 below.

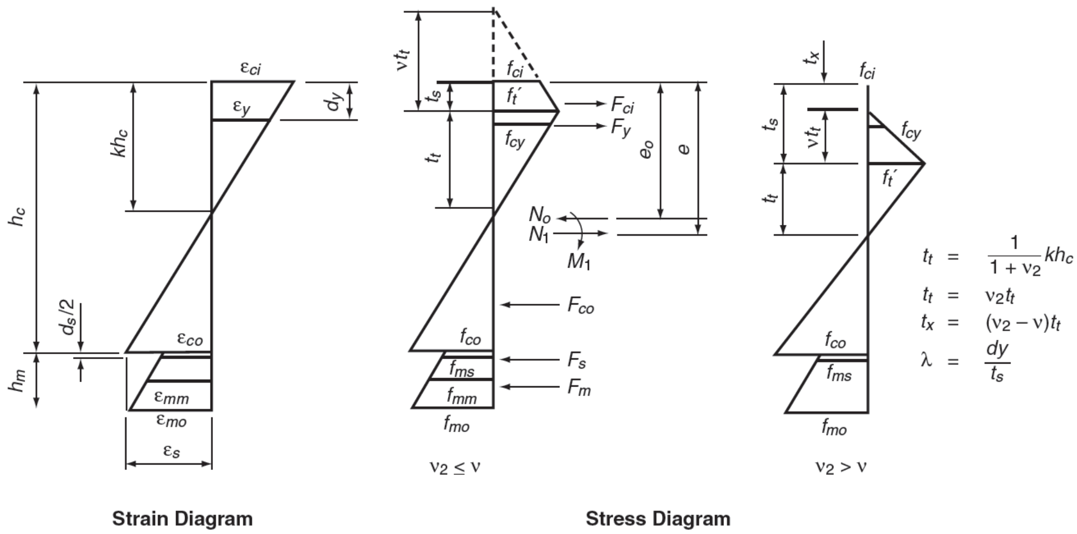

Figure 1. Schematic of strain and stress distributions in pipe-wall cross section at invert and crown (Figure 5 in AWWA C304)

Based on Figure 1 (Figure 5 in AWWA C304), it is apparent that most material stresses are based on linear behavior and, therefore, can be determined by simply multiplying the material strain times the modulus of elasticity of the material. As one example of potential simplification, consider the following equation provided in Section 8.9.1 of the C304 Standard for calculating the stress in the cylinder relative to the state of decompressed core concrete (Δf_y):

$$\Delta f_y = n' f_t' (1 + \upsilon_2) \left(1 - \frac{\lambda_y}{k}\right) \qquad \text{(Eq. 1)}$$

If we make the following substitutions for the terms in Equation 1:

$$n' = \frac{E_y}{E_c}$$

$$(1 + \upsilon_2) = \frac{\varepsilon_{ci}}{\varepsilon_t{}'}$$

$$\left(1 - \frac{\lambda_y}{k}\right) = \frac{\Delta\varepsilon_y}{\varepsilon_{ci}}$$

The result is as follows:

$$\Delta f_y = \left(\frac{E_y}{E_c}\right) f_t{}' \left(\frac{\varepsilon_{ci}}{\varepsilon_t{}'}\right) \left(\frac{\Delta\varepsilon_y}{\varepsilon_{ci}}\right)$$

The result shown above can be reduced down to the following simplified equation:

$$\Delta f_y = E_y \Delta\varepsilon_y \qquad \text{(Eq. 2)}$$

Similar simplifications can be made to the following equations of Section 8.9.1:

Existing C304 Equation		Simplified Equation
$f_{ci} = (1+v_2)\, f_t'$	→	$f_{ci} = E_c\varepsilon_{ci}$
$f_{cy} = f_t'(1+v_2)\left(1-\frac{\lambda_y}{k}\right)$	→	$f_{cy} = E_c\,\Delta\varepsilon_y$
$f_{co} = f_t'(1+v_2)\left(\frac{1}{k}-1\right)$	→	$f_{co} = E_c\varepsilon_{co}$
$\Delta f_s = n f_t'(1+v_2)\left(\frac{1+\lambda_s}{k}-1\right)$	→	$\Delta f_s = E_s\Delta\varepsilon_s$
$f_{ms} = m\left[f_t'(1+v_2)\left(\frac{1+\lambda_s}{k}-1\right)-f_{cr}\right]$	→	$f_{ms} = E_m(\Delta\varepsilon_s-\varepsilon_{cr})$
$f_{mm} = m\left[f_t'(1+v_2)\left(\frac{1+\lambda_m}{k}-1\right)-f_{cr}\right]$	→	$f_{mm} = E_m\varepsilon_{mm}$
$f_{mo} = m\left[f_t'(1+v_2)\left(\frac{1+2\lambda_m}{k}-1\right)-f_{cr}\right]$	→	$f_{mo} = E_m\varepsilon_{mo}$

It is noted that with the exception of concrete core cracking, no nonlinear response is considered in equations for strain and stress defined in Section 8.9.1 of the Standard. The cylinder, wire, mortar and concrete in compression are all assumed to be linearly elastic. Therefore, the equations can be significantly simplified as shown above. Demonstrating equivalence of the above stress equations is a matter of simple but tedious algebraic operations.

Determining the pipe wall strains and stresses is an iterative process that requires initial assumptions for the values of v_2 and k followed by calculations of the corresponding strains and stresses and equilibrium checks using the resultant forces and moments illustrated in Figure 1 and the equilibrium equations:

$$\Sigma F = 0$$

$$\Sigma M = 0$$

The simplified equations demonstrate that use of the v_2 and k terms are not necessary for calculation of the material component stresses. These terms are only needed to calculate material strain values. Use of these terms in the stress calculations only makes these equations more cumbersome and less understandable to the user. It is also noted that one of the stress/strain equation sets (for ε_{mm} and f_{mm}) is not needed to evaluate section equilibrium and, therefore, could be eliminated. Similar simplifications can be made in the equations provided in Section 8.9.2 used to evaluate strains, stresses, thrusts and moments at the pipe springline.

It is also apparent that the C304 Standard does not provide clear and complete guidance regarding how these equations are to be applied to lined-cylinder type PCCP. Equations and associated schematics used to calculate the stresses and strains in Section 8.9 of the C304 Standard apply only to embedded-cylinder type pipe, and no explanation or guidance is provided regarding use with lined-cylinder pipe. It was also noted that the only design example provided in the C304 Standard pertains to embedded-cylinder-type PCCP.

SIMPLIFICATION OF PROVISIONS RELATED TO DESIGN MATERIAL PROPERTIES

Design material properties are presented in Section 5 of the ANSI/AWWA C304 Standard. These include properties of the core concrete, mortar coating, steel cylinder and prestressing wire. The majority of the content of Section 5 consists of provisions used to define properties of the core concrete; more specifically, provisions to be followed to determine multipliers for the concrete modulus of elasticity (C_E), creep factor (C_ϕ) and shrinkage strain (C_s). These provisions include the requirement that PCCP manufacturers perform standard tests on molded cylindrical test specimens to facilitate evaluation of the multipliers.

Recognizing that the previous design methodologies for PCCP made use of a set of assumed values for core concrete material properties, it is reasonable to question whether it is actually necessary to adopt new provisions that contribute to the overall complexity of the design method. In addition to increasing complexity, it can also be argued that the new provisions for determining core concrete properties rely on test data derived from concrete test specimens that are not likely to be representative of the concrete incorporated in the pipe core due to various circumstances.

According to the C304 Standard, each factory where PCCP is produced shall perform tests on molded cylindrical test specimens made using the concrete mix with the aggregate and cement to be used in pipe manufacture to determine certain concrete material properties. These test specimens are required to be molded and cured in accordance with ASTM C192 and tested to determine compressive strength (per ASTM C39), modulus of elasticity (per ASTM C469) and creep and shrinkage properties (per ASTM C512) at an age of 28 days. These measured properties are subsequently used to evaluate the aforementioned multipliers for modulus of elasticity (C_E), creep factor (C_ϕ) and shrinkage strain (C_s).

One significant concern related to the approach presented in the C304 Standard is that placement, consolidation and curing conditions for the concrete incorporated in the core of a typical PCCP section differs significantly from the molding and curing conditions specified in ASTM C192. According to the C301 Standard, the concrete in the cores may be placed by the centrifugal method, vertical casting method, radial compaction method or other approved method. Unless otherwise specifically permitted, the cores shall be cured by the accelerated curing method, the water curing method or by the combination curing method. According to ASTM C192, the standard cylinders are typically consolidated by rodding/taping, and then subsequently moist cured at 23°C ± 2°C.

There is an abundance of available technical literature to indicate that concrete properties such as modulus of elasticity, creep and shrinkage are influenced by manufacturing parameters (such as method of concrete placement and curing regime) as well as subsequent exposures conditions (including those related to loading history and environment). The conditions represented by the molded cylindrical test specimens used to evaluate the multipliers for modulus of elasticity, creep and shrinkage are in many ways not representative of the core concrete incorporated in a PCCP section. Under these circumstances, there is reason to question the overall approach specified in the C304 Standard and the results of subsequent implementation.

Perhaps a more pertinent question related to the multipliers for the concrete modulus of elasticity (C_E), creep factor (C_ϕ) and shrinkage strain (C_s) is whether or not they are appropriate and necessary. If we are to adopt the logic that utilization provides a more technically-sound approach to design, then, at a minimum, the methodology needs to be refined to provide data to better represent the concrete incorporated in a PCCP section. However, before attempting to do so, we should first ask ourselves what we are gaining from utilizing such a detailed approach in the first place.

Concrete Modulus of Elasticity Multiplier C_E. With respect to the C_E multiplier, it is noted that other structural concrete design standards and codes such as those published by the American Concrete Institute (ACI) and the American Association of State Highway and Transportation Officials (AASHTO) do not include provisions for making adjustments to design modulus of elasticity (E_c). These other design standards rely solely on equations for calculating concrete modulus of elasticity as a function of the minimum design compressive strength (f'_c) and the design unit weight of concrete (γ_c) without use of a modification factor. Recognizing that use of standard equations for calculating modulus of elasticity for concretes with conventional strength levels have served the design community well for decades, adopting an alternative approach that would add unnecessary complication to the overall design process seems unwarranted.

The perceived need for a modulus of elasticity multiplier (C_E) may likely have been influenced by an issue that was identified in certain PCCP manufactured in a

Southern region of the US where relatively-soft limestone aggregates are prevalent. When used, these aggregates tend to result in concretes having lower density and overall stiffness than other concretes made using harder aggregates. However, based on results from tests performed by the author on core samples extracted for PCCP incorporating concretes with such aggregates, it is apparent that the equation for E_c currently provided in the C304 Standard (without modification) still provides an adequate estimate of modulus of elasticity. Hence, there are data to indicate that modification of E_c is not necessary even when concretes containing relatively soft aggregates are considered. It is also noted that the ASTM C33 Standard (to which all aggregates used in PCCP must conform) provides additional assurance against use of low-quality aggregates in PCCP. As an initial step toward simplification it can be argued that evaluation and application of the C_E term is not necessary for the reasons cited herein.

Concrete Creep Factor and Shrinkage Strain Multipliers (C_ϕ and C_s). According to the C304 Standard, creep and shrinkage deformation measurements obtained from cylindrical test specimens at an age of 28 days are to be extrapolated using one of two existing theoretical models (either the BP-KX or ACI 209R-92) to obtain what are thought to represent values that would be obtained at an age of 50 years. These values are designated as $\phi(18{,}250)$ and $s(18{,}250)$, respectively. The creep and shrinkage multipliers (C_ϕ and C_s) are subsequently obtained by dividing the theoretical 50-year values by what are stated to be the creep factor and shrinkage strain values obtained using ACI Committee Report 209R-92. These values are 2.0 and 700, respectively.

In a pipeline, perhaps more so than any other structure, conditions related to construction sequence, in-service exposure and operation can vary considerably. It is not uncommon for PCCP sections to remain in the plant or stored on site for several months before finally being installed. Once installed, it is not uncommon for a pipe to remain unfilled and unpressurized for long periods of time before finally being commissioned into service. Internal and external exposure conditions can range from a constant state of critical saturation, to periods of intermittent wetting and drying, to a constant state of relatively dry conditions over the length of a given pipeline. Operating pressures and external load effects are also expected to vary over the length of a given pipeline. When all of these potential variables are considered, along with the circumstance that many of these variables cannot be reliably accounted for during the design stage, it can be concluded that attempts to optimize or fine tune certain aspects of design are not warranted given the uncertainties involved.

The provisions cited in the C304 Standard for evaluating the design creep factor and design shrinkage strain constitutes one aspect of the overall design methodology where the complexity and corresponding level of effort required is not justified or warranted. To go to such lengths to calculate these two design parameters implies a level of precision that simply does not exist in the real world. The provisions contained in Section 5 pertain only to the creep factor and shrinkage strain multipliers (C_ϕ and C_s). The design creep factor (ϕ) and shrinkage strain (s), to which the multiplier may or may not be applicable, are defined elsewhere in the Standard. As an initial step toward simplification it can be argued that evaluation and application of the C_ϕ and C_s terms is not necessary for the reasons cited herein. Discussion of the design creep factor (ϕ) and shrinkage strain (s), along with proposed simplified equations, will be the subject of a future article.

CONCLUSIONS

Upon initial review, it is apparent that some of the PCCP design provisions currently incorporated in the ANSI/AWWA C304 Standard can be simplified without any significant corresponding consequences relative to the final overall design solution. Reducing the equations contained in Section 8.9 to a more simple form is one example. Simplifying provisions contained in Section 5 dealing with the evaluation of multipliers for the concrete modulus of elasticity (C_E), creep factor (C_ϕ) and shrinkage strain (C_s) is another example that is currently under study by the authors.

It also appears possible that other simplifications can be made throughout the C304 Standard. It is the intent of the authors to continue evaluating various other aspects of the C304 design procedure by studying the various references that are cited therein and performing parametric studies to identify specific parameters having a significant or controlling influence on the design outcome. The task of simplifying the C304 Standard will undoubtedly require a considerable amount of time and effort. Hence, the authors would welcome the assistance from various outside sources (academia, PCCP manufacturers, etc.) willing to help meet us this challenge.

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Cost Savings Using Optimization Methods for Water Conveyance Systems—Case Study for Recharge Fresno Program

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Abstract

The City of Fresno (City) serves approximately 513,000 residential customers and 35,000 commercial and industrial accounts. The City's water demands are approximately 145,000 acre-feet annually, which is met by 88% groundwater and 12% treated surface water. In order to address overdrafting of groundwater, the City has embarked on a major CIP that utilizes surface water allocations to reduce groundwater supply source from 88% to 36%. Construction of new 80 MGD treatment plant, several large diameter raw water pipelines, and approximately 40 miles of large diameter potable water regional transmission mains (RTM) has been recommended. The first phase of transmission mains initially included 25 miles of large diameter pipelines to connect to existing grid mains and distribution piping. Schematic design of the initial route for these and other alternative pipeline routes was completed concurrent with the optimization study. The approach to the optimization study was to take the information developed during schematic design and perform repetitive hydraulic modeling to determine the lowest cost alternative. The initial budget for the first phase of the CIP which included 25 miles of 48-inch to 24-inch RTM was estimated at about $88 million. After completing in excess of 700,000 hydraulic model runs, the optimum solution resulted in a recommendation of 13 miles of 66-inch to 24-inch pipe with a total cost estimate of $43M. The total cost to perform this optimization analysis was about $150,000 and resulted in about $45M in cost savings without compromising service objectives.

BACKGROUND

The City of Fresno (City) has a service area that covers nearly 114 square miles and serves approximately 513,000 residential customers and 35,000 commercial and industrial accounts. The City's water demands are approximately 145,000 acre-feet annually, which is met by 88% groundwater and 12% treated surface water. Groundwater production is achieved by utilization of 270 active municipal wells, while treated surface water is provided by the City's existing 30-million gallons per day (mgd) Northeast Surface Water Treatment Facility (NESWTF).

Although the City has an existing surface water treatment facility and an aggressive intentional groundwater recharge program, groundwater overdraft problems continue resulting in a decreasing groundwater table. In order to address this overdrafting of groundwater, the City developed what is known as the Metro Plan, which includes utilizing surface water allocations to reduce groundwater supply source from 88% to 36%. The Metro Plan includes construction of new 80 MGD treatment plant known as the Southeast Surface Water Treatment Facility (SESWTF), several large diameter raw water pipelines, expansion of the existing Northeast Surface Water Treatment Facility (NESWTF) from 30 to 60 MGD, and approximately 40 miles of large diameter potable water regional transmission mains (RTM). The first phase to be implemented consists of the 80 MGD SESWTF and 25 miles of Priority 2 pipelines to convey water from the new SESWTF. The next phase will follow about five years later with construction of the remaining 15 miles of RTM pipeline (known as Priority 3) to convey water when the NESWTF is expanded from 30 to 60 MGD.

Given the fact that the existing 270 wells are distributed uniformly throughout the City, there has not been a need to have large diameter pipelines to convey potable water. As a result of each well serving nearby demands the largest pipes in the existing system are 16-inches in diameter, with most pipes being 12-inches in diameter and smaller. The City has classified pipes 10-inches and smaller as distribution pipe, while 12-inch to 16-inch are known as transmission grid mains (TGM). The City of Fresno street system is essentially a large rectangular grid with major streets every mile and arterial streets every one-half mile. TGM pipes are typically found every one-half mile in the major and arterial streets. Figure 1 shows the general layout of the pipes and wells in the City, as well as the location of the existing NESWTF and the proposed SESWTF.

OBJECTIVES

Although there are a number of projects that will be implemented, the focus of this paper is on what are termed the Priority 2 RTMs since these are included in the first set of projects to be implemented with construction of the SESWTF. These include 25 miles of large diameter pipes that encircle the southern part of the City and will be used to convey water from the proposed 80 MGD SESWTF to the existing distribution system. Figure 2 shows the initial route of the Priority 2 RTM as depicted in the Metro Plan.

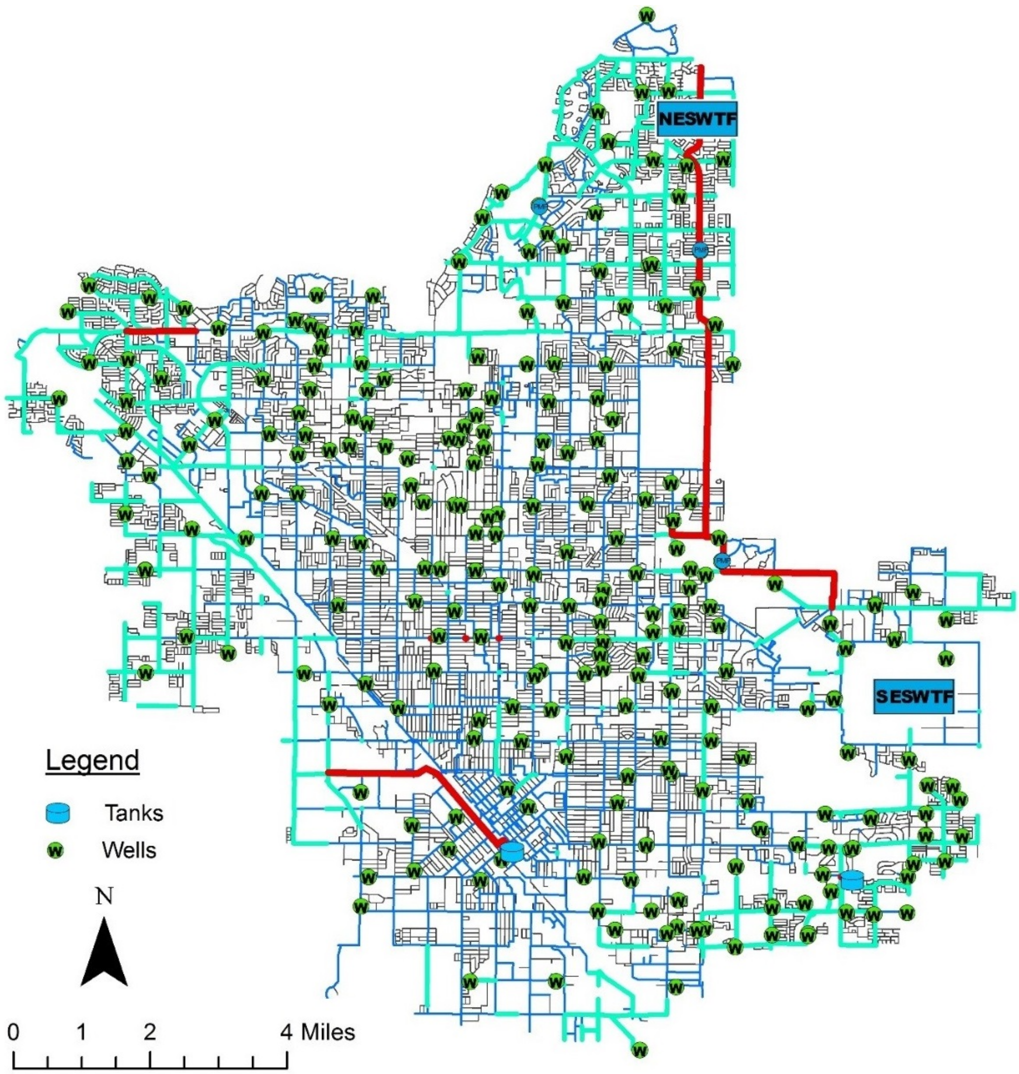

Figure 1 – Layout of City of Fresno Water System

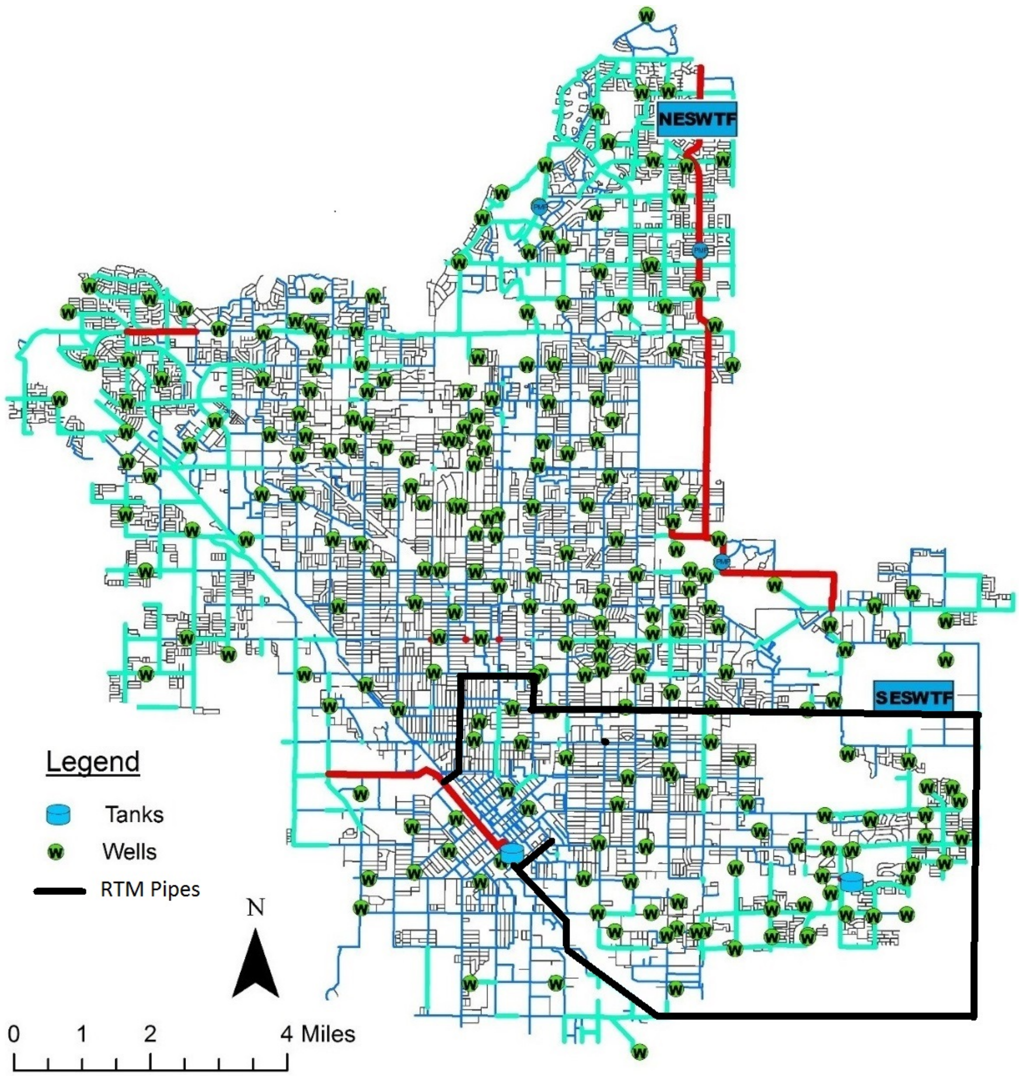

Figure 2 – Location of Priority 2 RTM from Metro Plan

The focus of this initial effort was to analyze these proposed routes, perform detailed hydraulic modeling to confirm pipe sizes and connections, develop possible alternative routes, and then perform optimization of the proposed and alternative alignments to minimize cost without sacrificing service to consumers.

OPTIMIZATION MODEL ARCHITECTURE

An optimization approach follows similar steps to a traditional analysis approach. Ultimately, the optimization is helping the user find a near optimal solution through an iterative approach. The optimization was performed using Innovyze as the hydraulic modeling software and a customized Excel spreadsheet to modify parameters and collect output for each scenario.

In this case, pipeline sizes and well on/off status are changed in each scenario. The optimization model generates a new string of options which are then sent to a base hydraulic model engine and run. The results are then imported into the evaluation module to produce a score made up of capital cost, energy cost, and penalty cost called the "Objective." The optimization module then analyzes the result to produce a new set of pipe and pump options and the process is repeated

INITIAL ANALYSIS

Parallel efforts were initially undertaken to drive towards the final objective of optimizing this system. One effort was to refine the hydraulic model of the system, while the other was to perform traditional field investigations to determine a final pipeline alignment.

Hydraulic Modeling

The City's hydraulic model was used to develop and recommend the initial pipeline routes and pipe sizes presented in the Metro Plan. This initial model was updated to reflect recent changes and used for the optimization.

The following demand scenarios were added to the hydraulic model and used in the optimization effort:

- *2020 Average Day Demand of 145 MGD*. This demand condition was used in conjunction with Priority 2 RTM Improvements. In these scenarios, the production of the existing NESWTF is at the current 30 MGD and the production of the future SESWTF is at 80 MGD.
- *2025 Average Day Demand of 159 MGD*. This demand condition was used in conjunction with Priority 3 RTM Improvements. In these scenarios, the production of the existing NESWTF is expanded to 60 MGD and the production of the future SESWTF is at 80 MGD. Similar to the 2020 scenarios, the groundwater wells supply the balance between the demand condition and available surface water treatment facilities.

The performance and design criteria are used to evaluate and judge the capacity adequacy of existing water distribution facilities and recommending improvements. The criteria includes the minimum acceptable customer service pressures during peak hour demands (PHD), maximum day demands (MDD), as well as during average day demands (ADD) and also includes the maximum allowed velocity in RTM and TGM pipelines.

- The desired minimum pressure during peak hour and maximum day demands is 40 psi
- The desired minimum pressure during average day demands is 50 psi
- The maximum pressures criteria for either demand scenario is 80 psi.
- The maximum desired velocity is 5 feet per second (fps) in both RTMs and TGMs.

Results of Initial Hydraulic Analysis

The initial RTM alignments shown previously in Figure 2 included a looped alignment along Olive Avenue on the north, Palm Avenue on the west, North Avenue on the south, and Temperance Avenue on the east. The Olive and Palm Avenues alignments are generally referred to as the northern portion of the loop (northern loop), while Temperance and North Avenues are referred to as the southern portions of the loop (southern loop).

The Metro Plan proposed connections from the RTM to the existing TGM approximately every 1 ½ miles, and each turnout was intended to include a meter and pressure regulating valve. Several hydraulic modeling scenarios were completed to determine the sensitivity of increasing the number of connections to the TGM (every ½ mile) as well as eliminating the associated meter and pressure regulating valves at each connection. The analysis indicated that the pressure regulating valves were restricting flows and limiting the conveyance capability of the Priority 2 RTMs. The analysis also indicated that increasing the turnout interval to every one-half mile resulted in a reduced pumping pressure at the SESWTF. Additionally, the half mile turnout intervals helped reduce some of the Priority 2 RTM diameters without increasing the velocities in the existing TGMs. Consequently, it was decided to increase the turnout intervals to every one-half mile and to exclude pressure regulating valves and flowmeters at these turnout connections.

These decisions resulted in reducing the length of pipeline from 25 miles shown in the Metro Plan to about 13 miles. In addition, it eliminates the need to construct additional TGMs in the southern part of the City.

Alignment Evaluation

The City hired a Consultant to provide preliminary design of the Priority 2 RTM, which included evaluation of several alternative alignments to those shown in the Metro Plan. Although the original scope of work was to develop alternative alignments for all 25 miles of the Priority 2 RTM, the focus of this effort shifted to alternative alignments for only the 13 miles in the northern portion of the Priority 2 RTM recommended as an outcome from the initial hydraulic modeling.

Evaluation of the Metro Plan alignment and alternative alignments were based on design information obtained including utility plats, potholes, geotechnical borings, traffic control requirements, permitting requirements and other factors. The objective was to confirm that the 13 miles of various size pipelines developed in the hydraulic modeling could be constructed in the alignments evaluated. The conclusion indicated that most of the alignment shown in the Metro Plan could be used, as well as one alternative for a portion of the 13 miles of RTM. A majority of the 13 miles of pipe do not have a viable alternative based on preliminary investigations that looked at traffic, density of other utilities, significant increase in pipeline length, and other constraints. However, there are two alignments that were carried forward for further consideration including 1) McKinley versus Clinton and 2) Chestnut. The location of the alignments included for further evaluation and optimization are shown in Figure 3.

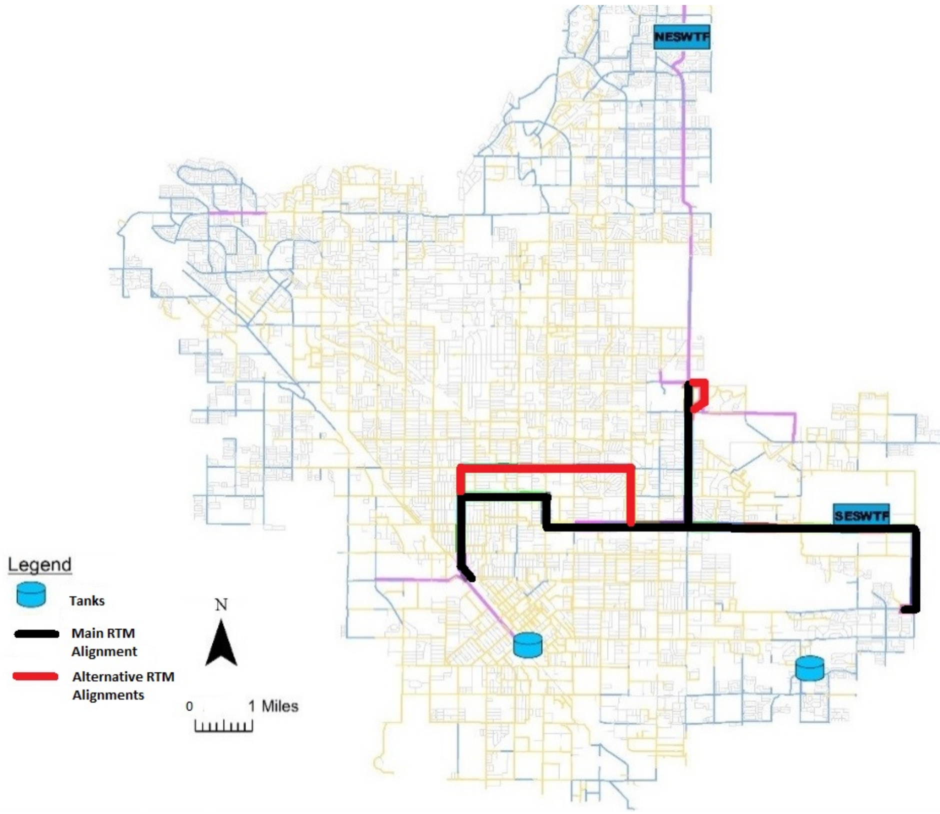

Figure 3 – Summary of Alignments for Further Evaluation

OPTIMIZATION INPUT

One of the first steps in the optimization effort is to segregate the entire pipeline into smaller segments commonly referred to as decision variables. Each of these decision variables was developed based on common features developed by the preliminary design team during their assessment of each alignment. Some of these common features include such items as pipe diameter, difficulty of construction, traffic requirements, and other features.

Once these decision variables were developed, then each was evaluated using both economic and non-economic characteristics.

Economic Characteristics

The primary economic factor to include in the optimization model was pipeline cost. The approach was to develop production rates for pipeline installation including Low, Medium, High, or trenchless. The low production rates are associated with streets that have a high concentration of utilities, while the high production rates are associated with more rural roads (fewer buried utilities).

Non-Economic Characteristics

Non economic criteria were developed and used in evaluation of the alternatives. Since the optimization model equates everything to cost comparisons, if a non-economic criteria was determined to be present for a segment, then an additional capital cost was assigned to that segment based on a percentage of the capital cost. The capital cost percentage assigned to each criteria are shown above in parenthesis after each criterion.

- Impacts to community (2%)
- Impacts to sensitive receptors (5%)
- Right-of-way or easement purchases (10%)
- Permit Costs (1%)
- Impacts from future improvements such as road widening, storm drainage, and underground projects (2%)
- Environmental documentation amendments (1%)
- Environmental remediation requirements (10%)
- Reductions of service of the General Plan area (5%)

Based on information obtain by the preliminary design team, each segment was evaluated to determine if each non-economic criteria was a consideration, For example, if a segment was aligned in front of a firehouse, then this was considered a sensitive receptor.

Penalty Costs

In order to account for modeling scenarios that violate the minimum and maximum criteria for pressure and velocity described earlier, penalty costs were assigned to that specific scenario. It was also important to target the discharge from the SESWTF to the RTM at 80 mgd, so there was also a penalty assigned to violating this criterion.

For example, if the minimum pressure is 50 psi, but a junction has a pressure of 49 psi, a small penalty is created. However, a pressure of 45 psi will generate a larger penalty. This difference in penalties is due to the calculation of the penalty function used, which is non-linear, as shown below:

$$Penalty\ Cost_{Pressure} = \sum_{i=1}^{n} A\left(|P_i - P_{boundary}|\right)^{2.5}$$

Where:

$Penalty\ Cost_{Pressure}$	=	penalty cost of pressure violation above or below pressure boundary
A	=	Scaling factor at node i, generally constant across nodes, but can vary
P_i	=	pressure at i-th node
$P_{boundary}$	=	Target pressure envelope (max pressure if pressure exceeded, min pressure if too low)

Per discussion with the City, a minimum pressure of 50 psi and maximum pressure of 80 psi was set as the boundary pressures. Scaling factor, A, was set at A = 5,000 for <50 psi, and A=300 for >80 psi, to further increase the penalty imposed on low pressures, as compared to high pressures. These scaling factors are variable and input based on experience from other projects and the user. A total of 110 junctions distributed throughout the system were chosen as the pressure penalty junctions.

Energy Cost

To include energy cost in the optimization, a life-cycle cost analysis was completed to estimate the total energy cost over a specific period. The objective of including energy cost was to create a balance between energy cost and the pipeline capital cost. Smaller pipe would result in a high hydraulic grade from the SWTFs, and larger pipe will reduce the energy costs. An analysis period of 50 years was selected for this life-cycle analysis based on a good balance between a typical pump's expected life (15 to 25 years) and a pipe's expected life (up to 100 years).

The NPV cost per surface water plant, or operational cost, was calculated as follows:

$$NPV_{50} = \sum_{n=0}^{50} P/F(i\%, n)(e_n kW_{annual})$$

Where:

NPV_{50} = net present value of energy cost to pump finished water from a surface water treatment plant
P/F = calculate present value give cost of future value of energy cost
i% = discount rate, 3%
n = nth year, up to 50
e_n = energy cost each year, escalation assumed to be 2%/year, starting at \$0.109 kWh/year for year 0
kW_{annual} = estimated kW used by a SWTF per year

$$kW_{annual} = \left[\frac{SWTF_{gpm} SWTF_{psi}\left(\frac{2.307\ ft}{psi}\right)\frac{0.746\ kWh}{1\ HP}}{3956 * \eta}\right] \times \left[\left(\frac{24\ hrs}{day}\right)\left(\frac{365.25\ days}{year}\right)(\%\ runtime)\right]$$

Where:

kW_{annual} = estimated kW used by a SWTF per year
$SWTF_{gpm}$ = full capacity of SWTF
$SWTF_{psi}$ = simulated discharge pressure at SWTF from optimization run
η = estimated pump efficiency, assumed 80%
% runtime = estimated percentage of time a SWTF will be pumping at full capacity

Summary of Costs

The optimization calculates all the costs previously described for each scenario and then adds them together using the following algorithm to develop a comparison cost:

$$TOTAL\ COST = NPV_{50} + Cost_{Capital} + Cost_{NonEconomic} + Penalty\ Cost_{Pressure} + Penalty\ Cost_{Velocity} + Penalty\ Cost_{Flow}$$

RESULTS

More than 700,000 evaluations of the City hydraulic model for different scenarios were completed using the optimization tool including four of the top solutions to highlight the slight individual differences in desirable cost capital, energy cost, and hydraulic performance. Solution 1 is a “hybrid” of capital cost and energy cost, Solution 2 has the lowest energy cost, Solution 3 has the lowest capital cost, and Solution 4 has the lowest penalty cost. A cost summary for the starting solution and the top four solutions are provided in Table 1 below. In addition, the starting solution prior to optimization developed in the initial hydraulic modeling phase is included as baseline for comparison purposes.

TABLE 1
Top Solutions Cost Summary (cost in $M)

Solution	Const. Cost	Operational Cost	Pressure Penalty Cost	Velocity Penalty Cost	Capital + Operational Cost	Total Cost (including Penalty Costs)	Capital Savings from Starting Solution	Capital + Operational Savings from Starting Solution
Starting Solution	$50.59	$66.26	$0.01	$0.00	$116.85	$116.86	-	-
1	$42.86	$69.41	$0.06	$0.01	$112.27	$112.34	$7.73	$4.58
2	$43.15	$67.83	$0.53	$0.01	$110.98	$111.51	$7.45	$5.87
3	$42.42	$69.13	$0.11	$0.11	$111.55	$111.67	$8.17	$5.30
4	$42.42	$69.96	$0.01	$0.01	$112.39	$112.41	$8.17	$4.46

The four top solutions were selected to illustrate the diversity of workable solutions that produce similar low objective costs. All the solutions provided are feasible solutions with cost savings; and each has its own set of benefits. Following is a discussion of each of the top four solutions:

Initial Modeling Solution: This is a good solution if energy is thought to be more expensive than estimated. It allows for the lowest discharge pressure out of the SESWTF and, therefore, has the lowest operational costs out of all the solutions. However, it does have the largest capital cost.

Solution 1 - This is a “hybrid” solution of capital costs and operational costs. It was found to have minor penalties and a good balance between construction and operational costs. This solution also does not include pipe diameters larger than 66 inches, which is considered preferable for constructability purposes.

Solution 2 - If the pressure violations shown in the simulation results are acceptable, this solution allows for the lowest operational cost. This solution is largely energy cost-driven.

Solution 3 - This solution has the lowest construction cost, and it is also a good solution if energy is thought to be less expensive than currently estimated.

Solution 4 - This has the same construction cost as solution 3; however, this solution shows how the operational costs can vary depending on which supplemental wells are active.

CONCLUSION

Ultimately, Solution 1 was selected as the recommended project. The decision variable numbers and recommended pipe sizes are shown in Figure 4. The following characteristics of this solution were used to support this recommendation:

- Solution 1 has a good balance between construction cost and operational cost. It is not completely driven by future energy prices (higher or lower).
- Solution 1 does not result in minimum pressures below 45 psi anywhere in the distribution system. An "on" setting of 45 psi is typical for the wells in the system. Solution 1 does not need more wells turned on in order to maintain minimum pressures; only the ones selected in the optimization are needed.
- The maximum velocity observed in a RTM for Solution 1 was 5.1 fps. While this is above the criteria of 5.0 fps, it is only by a small amount. Any solution with RTM pipe velocities much larger than 5.0 fps are not desirable.

Figure 4 – Recommended Alignment

SUMMARY

The primary focus of this exercise was to minimize costs to implement this system without sacrificing the level of service required. Through the initial planning process by others, approximately 25 miles of large diameter pipelines were anticipated with a construction cost of about $88 million. Our first step was to obtain the hydraulic model used in the initial planning process and consider potential alternative scenarios based on new operating criteria. This exercise resulted in reducing the size and cost of these Priority 2 RTM from the initial 25 miles to 13 miles, with a corresponding reduction in the construction cost from $88 million to $50.6 million. The next and final step was to perform optimization of this reduced system by modifying combinations of pipe sizes with cost attributes for each pipe size. This exercise resulted with about the same length of pipeline (13 miles), but it identified optimized locations and pipe sizes at an even lower construction cost which is estimated at about $42.9 million. The total construction cost savings, attributed to this optimization effort, is estimated at approximately $45 million.

Setting the Record Straight—ISO S4 Testing for AWWA C900 Pipe

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Abstract

S4 testing per ISO 13477 is a lab-based procedure to determine the critical pressure above which a fracture will propagate in a given pipe material after initiation. The test was developed for the natural gas industry using air as the test medium because only compressible fluids can provide enough energy to propagate fractures after an initiation event. Recent industry discussions have focused on water applications as well. A small amount of testing using water was performed in the 1990's on PVC pipe extruded in the UK to the standards in place at that time. Until now, no testing had been completed on AWWA C900 PVC pipe using the S4 test method as prescribed in ISO 13477. Instead of speculating with outdated and ill-fitting data, actual testing has now been done. The test results show the differences in results between AWWA C900 PVC currently extruded in North America and the PVC pipe on which some testing had been done in the UK in the 1990's. This paper discusses the test methodology, test results, and the accurate and updated conclusions for modern PVC pressure pipe made to AWWA C900.

INTRODUCTION:

Rapid crack propagation (RCP) is a fast long running fracture. RCP is a post-failure event requiring an initiation of a fracture and an energy source to drive the fracture along the pipe wall in the direction of the pipe's longitudinal axis. This behavior, being detrimental to the integrity of the piping system has been known to occur in all types of pipe in pressure applications, including steel pipe [1] and HDPE. A 30" DR 9 HDPE water line was rehabilitated earlier this year to remedy a long gradually spiraling longitudinal crack. The slip line covered the entire 2800' length

To overcome the impractical burden of performing full scale RCP testing, a laboratory based test method was developed in the 1990's to determine the critical pressure (P_c) for rapid crack propagation in a given pipe material. This laboratory scale test method is known as the small scale steady state, or S4 method. $P_{c,\,S4}$ is the on-set pressure above which a crack will propagate and below which the crack arrests in the test method. An International Standard for this test method has been developed and implemented that describes the experimental set-up, procedure, and basis for scaling the S4 test result to a full scale critical pressure value [2].

While the S4 method (ISO 13477:2008 Thermoplastics pipes for the conveyance of fluids—Determination of resistance to rapid crack propagation(RCP)—Small-scale steady-state test(S4 test)), was developed primarily for pipes conveying natural gas where RCP was a concern, the standard allows the method to be applied to any other fluid being conveyed including water.

There have been only a few reports on the use of S4 testing for PVC water pipes. In the 1990's Greenshields and Leevers published several papers [3, 4] on testing with water and water/air medium combinations in UK PVC pipe made to the BS 3505 standard. More recently some limited testing has been performed on PVC pipes in the Netherlands [5]. The Netherlands-tested pipe was determined to have been over-gelated during extrusion. In both cases the test-set up was modified from that prescribed in ISO 13477. In the case of the UK tests, the S4 tests were performed with the baffles completely sealed to the inner pipe wall with rubber rings. In the Netherlands testing, baffles were not used at all.

In the present work, critical pressures on various sized (outside diameter and dimensional ratio) PVC pipe were determined per ISO 13477. The PVC specimens were manufactured to the specifications for sale under the Fusible PVC® brand. In addition, experimental measurements of factors that are known to affect RCP in pressure pipes are described in terms of crack velocity and decompression wave speed. Further, some information on the effect of temperature is provided as well as the influence of air volume in the pipe on the RCP measurement P_c.

EXPERIMENTAL SET-UP

An S4 test set-up per ISO 13477 as employed in this work is illustrated in Figure 1. The striker blade is made to travel at speeds between 10 m/s (meters per second) and 20 m/s. A photo-electric timing gate is placed on the impact frame in parallel to the line of impact to measure the striker speed. An external cage around the test specimen is made in specification to the standard and is sized to 1.1-x the pipe OD. To enable the S4 test to be performed with water medium, a capture basin and drain mechanism is placed under the S4 jig to capture the water from the pipe specimen during crack initiation and subsequent RCP. An anvil meeting the requirements of the test method is used along with the prescribed set of baffles. The baffles are spaced at 0.4-x the pipe OD and the diameter of the baffle is 0.95-x the pipe ID.

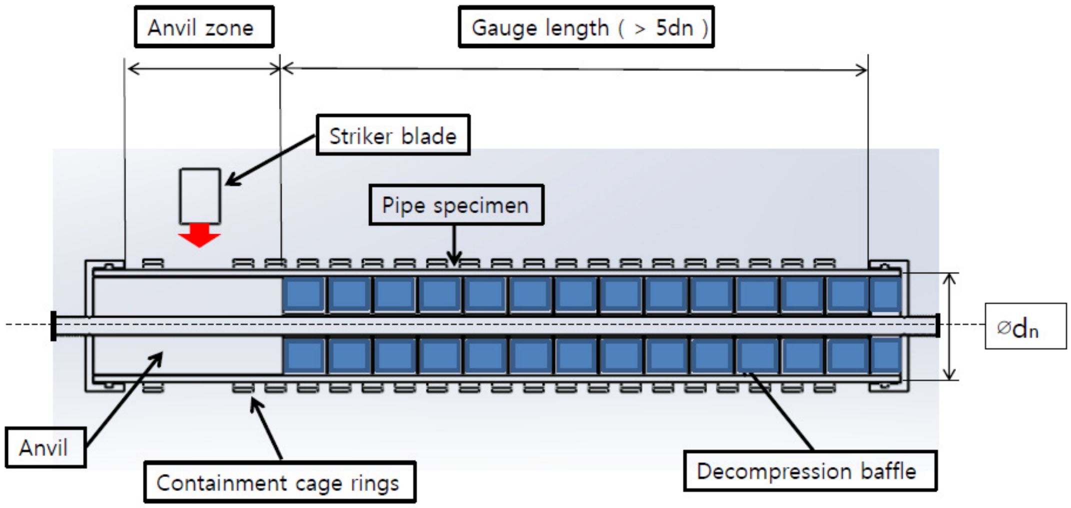

Figure 1. Diagram of the water medium S4 test set-up for PVC pipe

At the beginning of the test, the crack length with zero internal pressure was determined and it was confirmed that in all tests the crack length observed was above 0.7-x the pipe OD.

A set of tests at different pressures was performed per the standard. A group of six to eight is normally sufficient to define the test data. The test results were plotted to a ratio of crack length to OD versus the internal test pressure of each specimen. The test pressure was gradually raised until there was a steep jump in crack length (plotted as a multiple of the pipe OD. Once a crack length of ≥ 4.7-x pipe OD was attained, several tests were done at higher internal pressures to confirm this threshold. The internal pressure required to drive a fracture≥ 4.7-x pipe OD in the test configuration is considered the critical pressure or $P_{c,\ S4}$. At internal pressure above this value a crack will propagate and at or below this pressure arrest will occur.

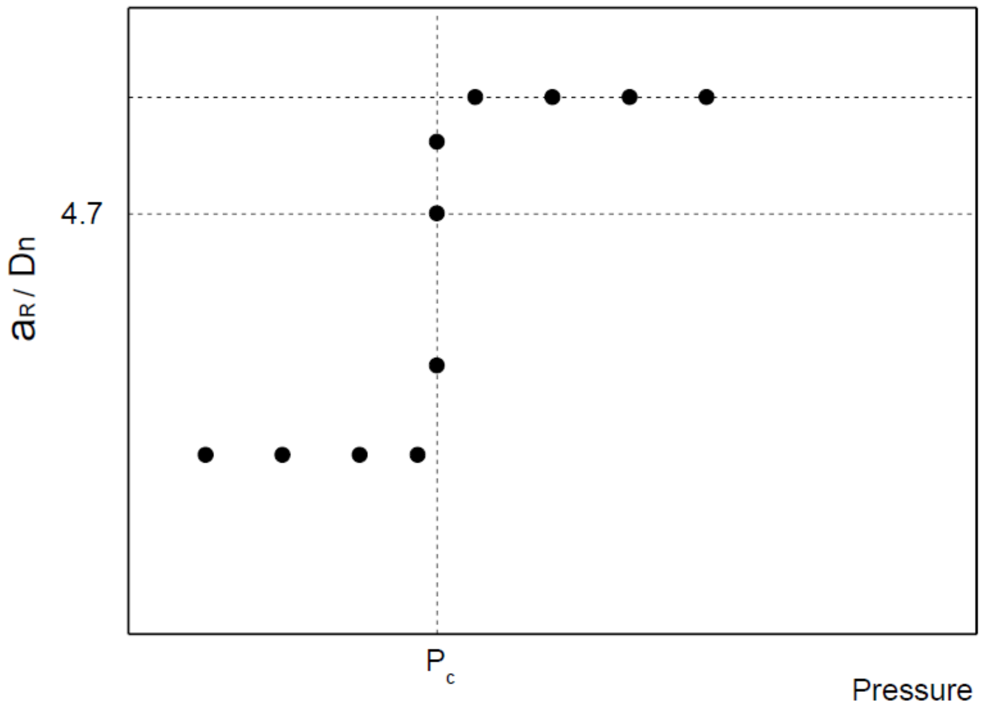

Figure 2. Example of S4 test results and $P_{c,\,S4}$

This pressure is the S4 test critical pressure. To determine how this translates to a full scale installation, ISO 13477 provides an equation to be applied to the S4 critical pressure to derive the full scale critical pressure. The equation is:

$$p_c = p_{c,FS} = 3{,}6p_{c,S4} + 2{,}6 \quad (1)$$

Where $P_{c,\,FS}$ and $P_{c,\,S4}$ are expressed in Bar.

In addition to the ISO 13477 test set-up, a data acquisition system was developed and used in conjunction with the ISO 13477 testing to determine crack velocity and decompression wave speed. The additional data points gathered include circumferential break wires placed at precise intervals on the test specimen. As a fracture moves along the test specimen, the wire separates allowing a voltage change to be recorded at the precise time interval. This results in data that provides an accurate crack speed for each test pressure.

A second set of data collection points in the form of pressure transducers were also installed. These transducers are placed at precise intervals to allow collection of pressure changes in the test specimen as a fracture moves along the specimen.

These instrumentation networks allow the capture of the data that is needed to determine crack velocity and water decompression wave speed.

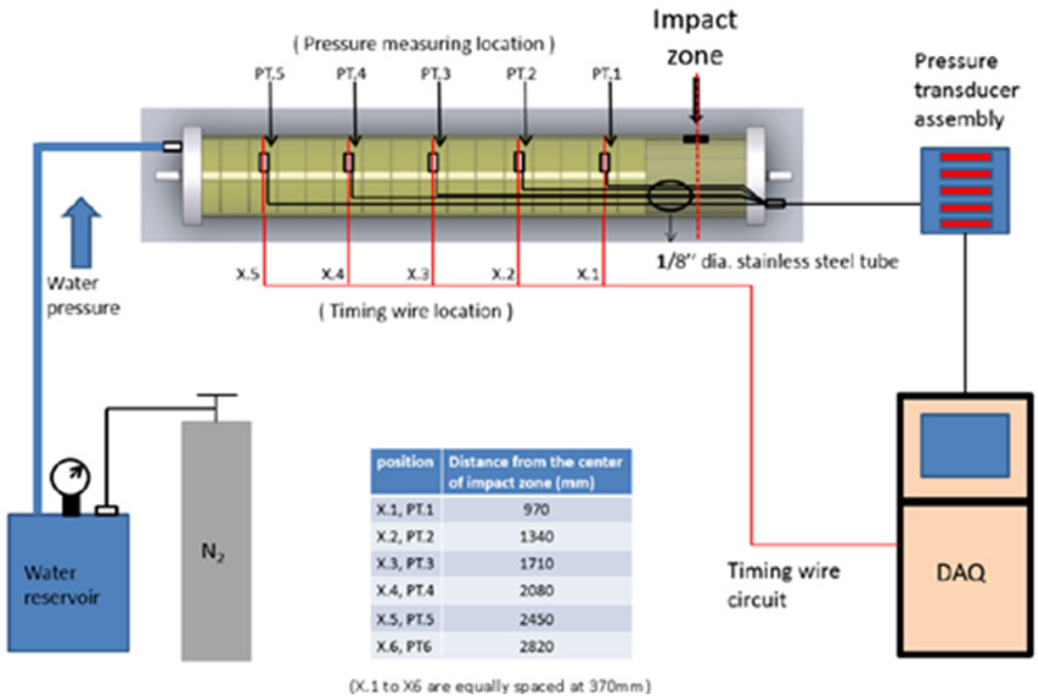

Figure 3. Data acquisition schematic

TEST SPECIMENS:

Pipe specimens tested were taken from commercial pipe lots manufactured to AWWA C900-07 Polyvinyl Chloride (PVC) Pressure Pipe and Fabricated Fittings 4 In. Through 12 In. (100mm Through 300 mm), for Water Transmission and Distribution. Pipe from each of the lots has been installed in municipal projects. The pipe lots from which the specimens were cut were also extruded at several extrusion locations.

Multiple diameters and wall thicknesses were included in the test specimen selection. The sizes selected for testing are commonly used by water utilities. The preferred sizing convention for municipal water pipe is ductile iron pipe size outside diameter. Several different diameters were tested. Wall thickness for municipal water pipe is determined by the ratio of outside diameter to minimum wall thickness. This defines the dimension ratio or DR of the pipe. Again, commonly used DR's were selected for testing.

In addition, for the 6" DR 18 test specimens, both pipe-only and pipe with a fused joint approximately centered in the specimen were used for testing. The fusions were performed by a qualified PVC fusion technician using a fusion machine meeting the minimum equipment requirements. The fusion procedure developed for fused PVC was used.

In accordance with ISO 13477, the length of each specimen was between 7-x OD of the pipe and 8-x OD of the pipe.

RESULTS AND DISCUSSION:

Initial test results for 6” (175 mm) DR 18 PVC pipe tested in accordance with ISO 13477 are as follows:

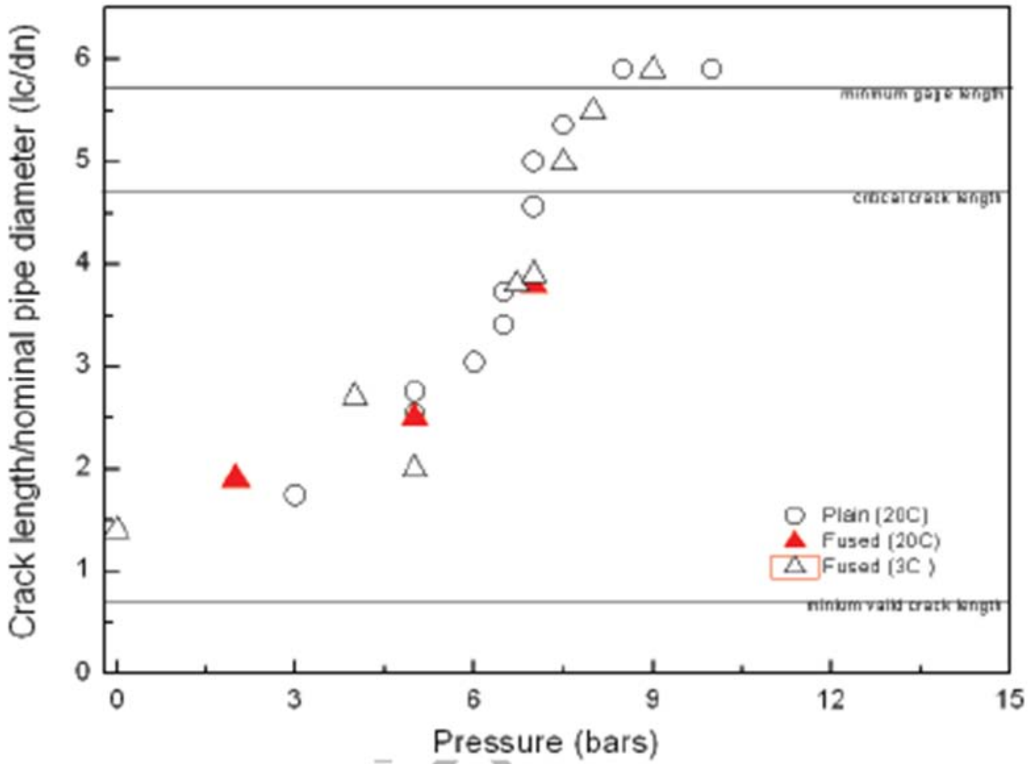

Figure 4 6” (175mm) DR 18 graph of crack length vs internal pressure

A set of 22 individual tests were performed. The crack initiation test was performed at no pressure, resulting in crack length greater than 0.7-x the pipe OD.

As the internal pressure is increased for each test, the crack length increases until reaching 4.7-x the pipe OD. The resulting $P_{c,\,S4}$ is 7 Bar (102 psi).

In Figure 4 above, the measure for the Y axis is the distance of the crack travel divided by the outside diameter of the test specimen. Both plain pipe and pipe with a fusion joint were tested.

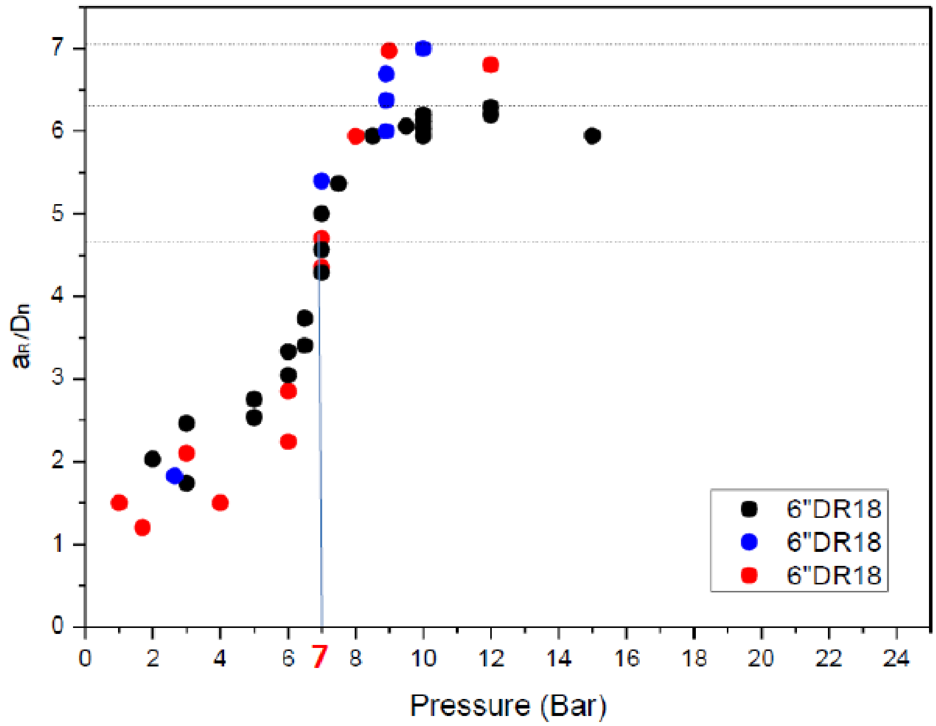

Figure 5. 6” (175mm) DR 18 results for different production lots for $P_{c,\,S4}$

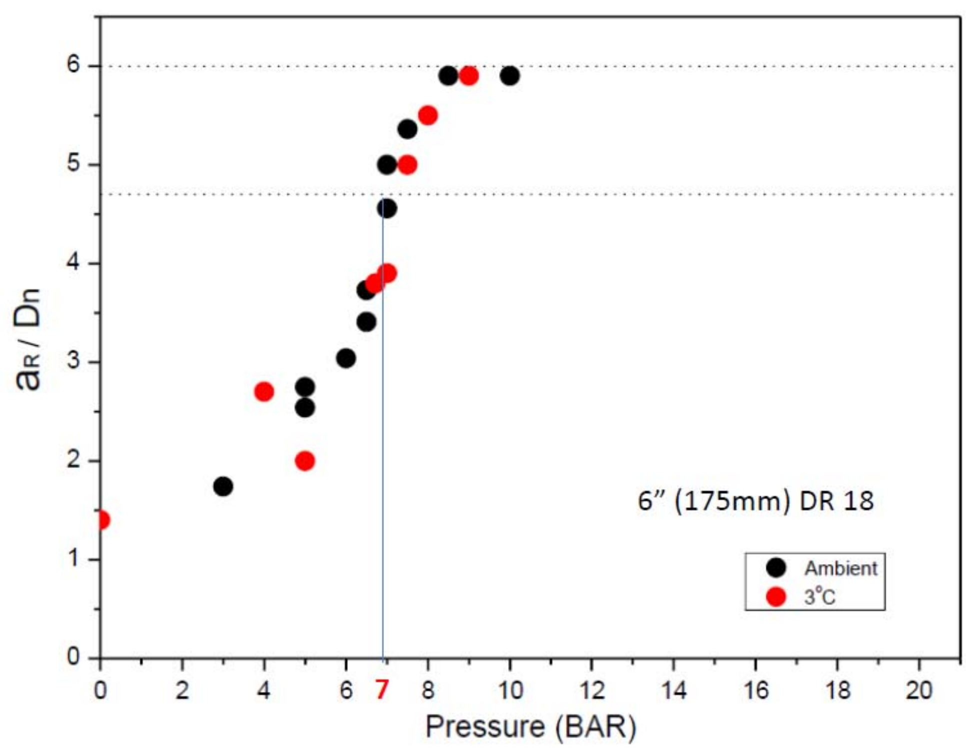

Figure 6. 6” (175mm) DR 18 $P_{c,\,S4}$ results at different temperatures

In Figure 5, several different lots of the PVC pipe tested are plotted with no discernable difference in the results.

Within the data set shown in Figure 6, temperature was also varied between 3° C and 20° C. No difference in results was observed between the two temperatures.

Another set of ISO 13477 S4 tests were performed on test specimens containing different combinations of water and air. A set of 60 tests were completed.

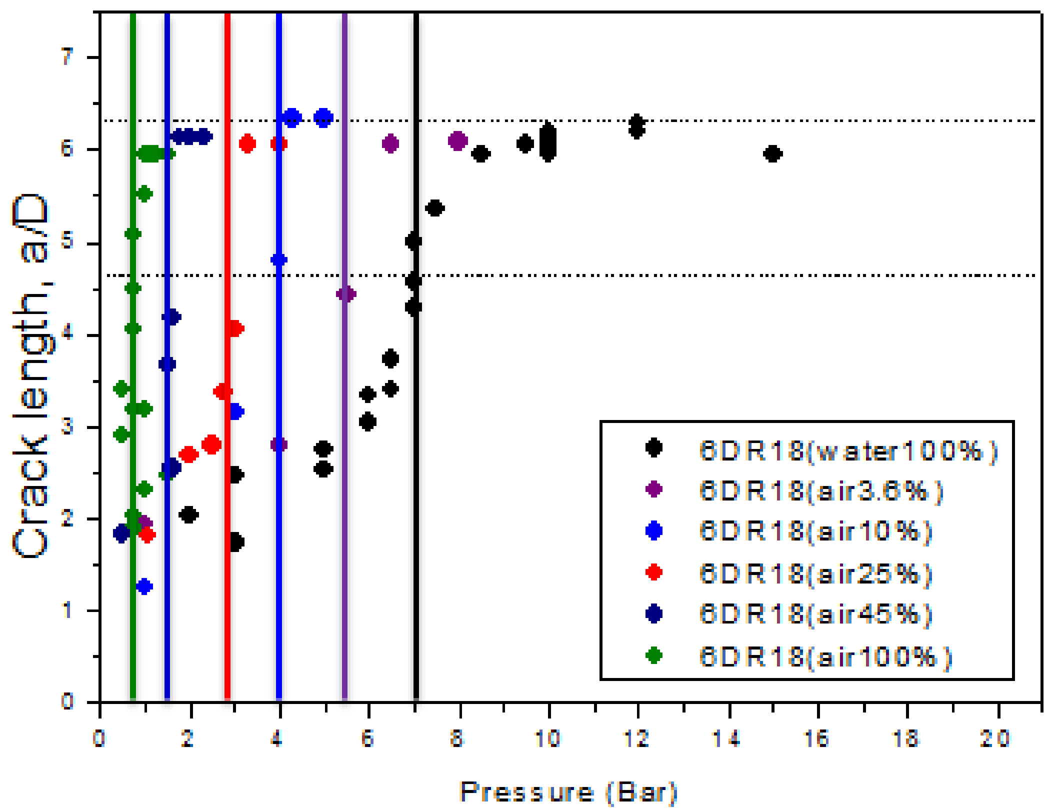

Figure 7. 6" (175mm) DR 18 graph of crack length vs internal pressure for various water to air combinations

The six subsets of data represent various water- to-air combinations. An S4 plot was made for each of these in Figure 7.

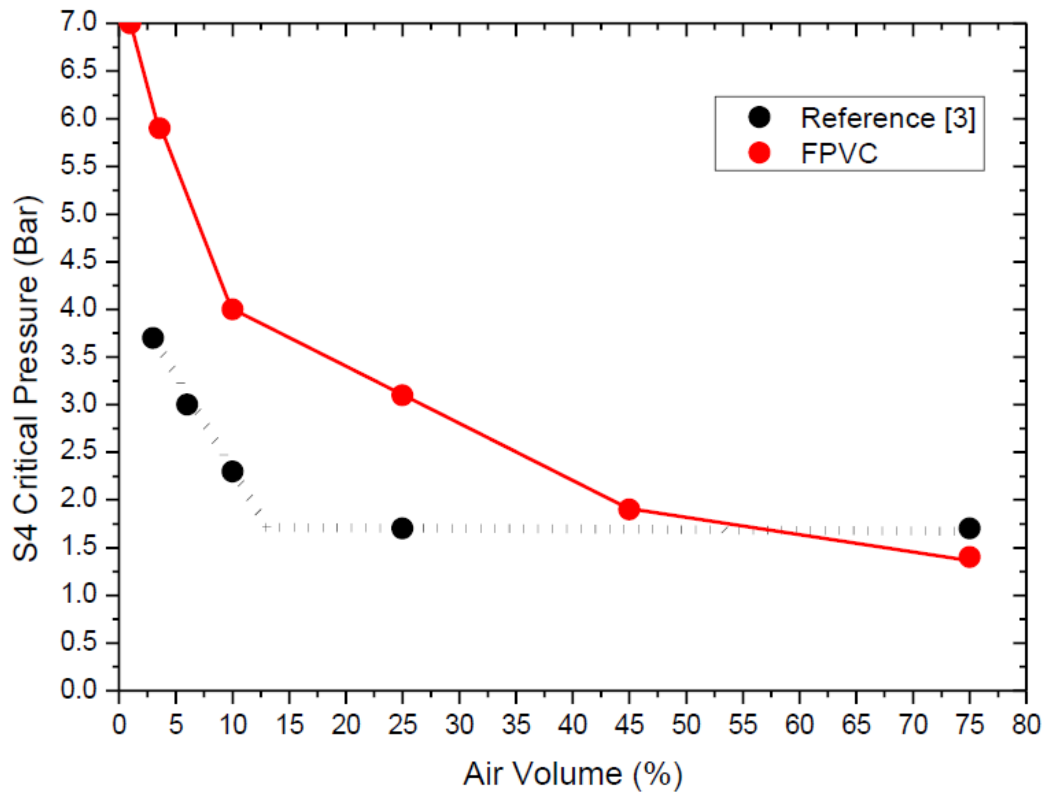

Figure 8. Plot of $P_{c,\,S4}$ vs % air volume test results of 114mm DR 19 PVC pipe manufactured in the U.K.[3,4] along with those for AWWA C900 6" (175mm) DR 18 PVC pipe (a reasonable comparison is provided using the the 6" AWWA size. It would be expected that a 4" AWWA size PVC test specimen would have the same, or possibly higher, results)

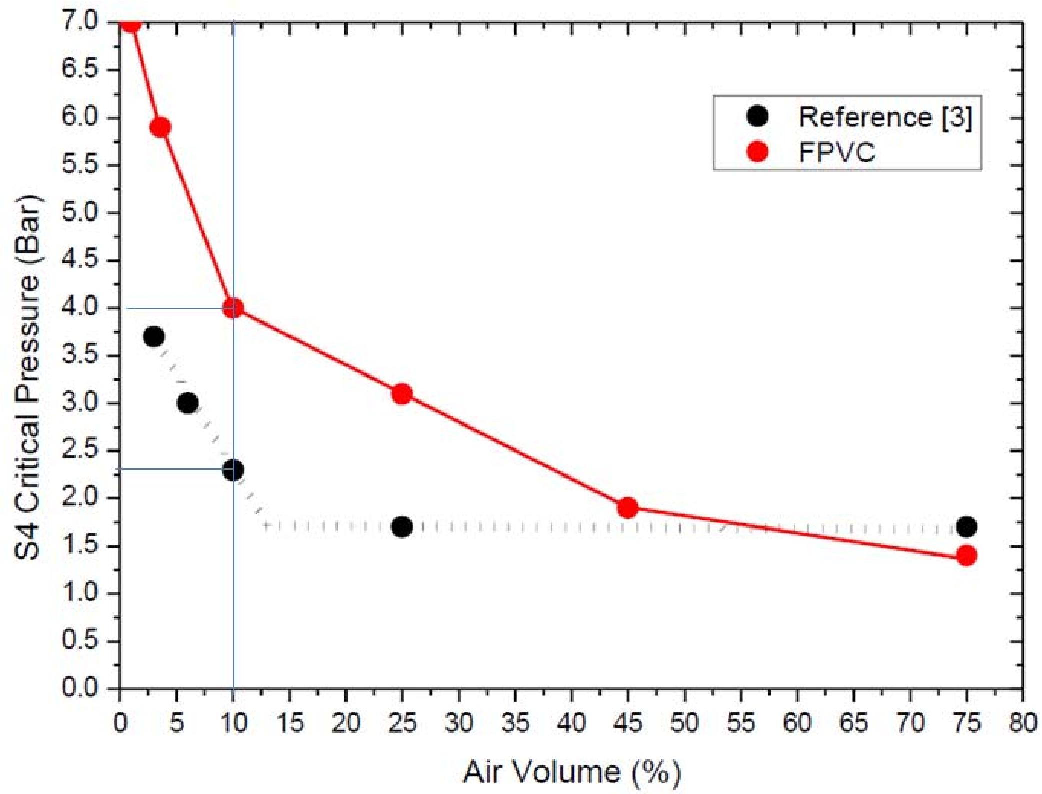

Figure 8a. Plot of $P_{c,\,S4}$ vs % air volume test results of 114mm DR 19 PVC pipe manufactured in the U.K.[3,4] along with those for AWWA C900 6" (175mm) DR 18 PVC pipe with the 10% air results delineated

Each of the $P_{c,S4}$ is then plotted against the percentage air by volume in the test specimen(Figure 8). Figure 8 also depicts the $P_{c,S4}$ derived from testing conducted in the 1990's on 114 mm DR 19 PVC pipe extruded in the U.K. to BS 3505-1986, the only other known data of this type. The 1990's testing was not, however, conducted in accordance with the ISO 13477, The baffles were fitted with rubber seals make direct contact with the test specimens' internal wall[4].

As internal pressure of each S4 test was increased, the crack speed increased, while decompression wave speed did not change and stayed approximately steady at a little over 200m/s. The crack speed measured at $P_{c,S4}$, is 350 m/s. Baffles required by the ISO 13477 test method were seen to slow the decompression wave speed down from a theoretical value reported at 484 m/s [4] to about 200 m/s for DR 18 pipes. It is to be noted that the criterion for crack speed being higher than the decompression wave speed for the rapid crack propagation to occur seems to be satisfied for the S4 test.

SETTING THE RECORD STRAIGHT:

S4 Critical Pressure Results and Full Scale Critical Pressure Results with PVC:

The critical pressure for S4 testing performed in accordance with ISO 13477 with water as the fluid as shown in Figure 5 for 6” DR 18 PVC pipe manufactured to AWWA C900-07 requirements is 7 Bar, or 101.5 PSI (using the conversion factor of 14.5 PSI per 1 Bar). The ISO 13477 equation to adjust the S4 critical pressure (in Bar) to full-scale critical pressure (in Bar) is:

$P_{C,\,FS} = 3.6(P_{C,\,S4}) + 2.6.$

With the $P_{C,\,S4}$ result for 6” AWWA C900 of 7 Bar inserted into this equation, the resulting full-scale critical pressure is 27.8 Bar or 403 psi.

The AWWA pressure class for 6” DR 18 PVC pipe is 235 psi.

Pressure rating of 114mm DR 19 PVC pipe:

In the testing done in the 1990’s in the UK, the PVC pipe tested conforms to the BS 3505-1986(British Standard). Publications describing the tests performed and BS 3505-1986 indicate the pressure rating for the 114 mm DR 19 PVC pipe was 12 Bar or 174 psi.

The AWWA C900-07 standard provides that the pressure class for compliant PVC pipe is determined by the following equation:

$$PC = \frac{2}{DR-1} \times HDB \times DF \text{ where:}$$

PC = Pressure Class	HDB = Hydrostatic Design Basis
DR= Dimension Ratio	DF = Design Factor

With a given pressure rating and using a design factor of 0.5, the HDB for the BS 3505-1986 pipe can be determined by rearranging the terms to:

$$HDB = PC \times \frac{DR-1}{2} \times \frac{1}{DF}$$

The result yields an apparent HDB for the 114 mm DR 19 PVC pipe of 3132 psi.

The HDB required in, and used by AWWA C900-07, in determining pressure class is 4000 psi.

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Full Scale Hoop Stress for AWWA C900 PVC Pipe:

Hoop stress(S) is the stress required in the PVC pipe wall for a given internal pressure. For the full scale critical pressure of 403 psi in 6” DR 18, the hoop stress is derived by the following:

$S = \frac{P(DR-1)}{2}$, where P = given pressure and DR = Dimension Ratio.

In this case, the pressure is 403 psi and the Dimension Ratio is 18. Solving for S yields a hoop stress value of 3425 psi.

The hoop stress for the 235 psi pressure class consistent with AWWA C900 is 2000 psi.

S4 / Full Scale Critical Pressure/Hoop Stress of Fused PVC pipe with 10% Air:

As shown in Figure 7 S4 tests were performed at different percentages of air content in the test medium of water. At 10% air volume, the S4 critical pressure is 4 Bar or 58 psi. Using the same full scale equation set forth above, the full scale critical pressure is 17 Bar or 246.5 psi. Following the previously explained methodology for hoop stress, for 10% air volume, the hoop stress is 2095 PSI.

Crack Velocity:

With the data acquisition system described in Figure 3, crack velocity measurements during S4 testing were made with water as the conveying fluid. Timing wires were attached to the test specimen circumferentially at evenly spaced intervals. As a fracture moved through the location of the timing wire, the wire would break causing a recordable change in voltage to the timing circuit. For 16” DR 18 PVC test specimens, multiple timing tests were done at different internal pressures. Crack velocity was 350 m/s at S4 critical pressure.

CONCLUSIONS:

AWWA C900 6” (175mm) fused PVC pipe, when tested in accordance with ISO 13477, has a full scale critical pressure of 403 psi. The designated pressure class for 6” (175mm) DR 18 PVC pipe is 235 psi.

All unplasticized PVC (PVCU) pipe is not the same in terms of performance and does not have the same performance criteria. PVCU pipe tested in the UK in the 1990’s displayed much different performance parameters than today’s AWWA C900 PVCU pipe. This is evident by the hoop stress and HDB values previously discussed.

S4 test results from PVCU with different performance parameters are different as well.

ACKNOWLEDGEMENT:

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Development of a Testing Protocol for Fatigue Testing of Large Diameter HDPE Pipes

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Abstract

The drinking water infrastructure in the North America requires a durable and reliable water transmission pipe material. However, there is no known standard to evaluate large diameter high density polyethylene (HDPE) under cyclic loads to investigate its fatigue performance, as it a major concern for water utilities. As part of a wider-scale research project to investigate durability and reliability of large diameter HDPE pipe (Water Research Foundation (WaterRF) #4485), a testing protocol was developed at the Center for Underground Infrastructure Research and Education (CUIRE) at the University of Texas at Arlington to test a large diameter (16 in. and larger) HDPE pipe under cyclic surge pressures. This paper presents details of test setup, and results of testing for a 16-in., DR 17, 15-ft long pipe sample with a fusion joint in the middle. The testing consisted of two phases. The pressures used for the first phase were between 125 and 188 psi (1.5 times pressure class) for 2 million cycles. The 2 million cycles are equivalent to 50 pressure surges per day for a 100-year design life. A second phase was later added using the same pipe sample to evaluate occasional surges between 125 psi to 250 psi (two times pressure class) for 50,000 cycles. The testing was completed with pipe sample's minor dimensional changes primarily due to limited relaxation allowed during the pressure cycles.

INTRODUCTION

Long-term steady pressure design and performance of plastic piping material is evaluated using ASTM D1598 (2009) and ASTM D2837 (2013). Design factors for long-term durability are established by the PPI's Hydrostatic Stress Board[1] (Boros 2011). The elevated temperatures and sustained pressure requirements for PE4710 material are addressed by ASTM F714 (2013) and AWWA C906 (2006) as well as PENT testing per ASTM F1473 (2013). While these studies indicate a high resistance to fatigue for HDPE, the data were gathered on small diameter pipes. However, testing is required for large diameter pipes to confirm the fatigue test results for all pipe sizes.

Reliable and durable water mains must have adequate resistance against recurring pressure surges to avoid fatigue failures. However, one area of durability that has not been thoroughly investigated is the fatigue resistance to recurring pressure surges for large diameter HDPE pipes. This paper will cover an experimental procedure to help in evaluating the reliability and durability of large diameter HDPE pipe.

Transient pressure variations commonly occur in water mains and transmissions lines during daily operations. Pump starts and stops and valve openings and closings can cause sudden and significant changes in flow. The amplitude and frequency of the resulting pressure variations (pressure surges) may affect the durability of the piping material. AWWA C906 permits frequent pressure surges to 1.5 times the pipe's pressure class (PC) and occasional pressure surges up to two times the pipe's pressure class. These factors are based on PE4710's short-term rupture strength with an understanding that a very large number of surges can occur in HDPE pipe during its design life. .

This paper is based on a research project to develop a testing protocol and execute a fatigue test on a 16-in. diameter, 15-ft, DR 17 with a butt-fused joint in the middle. The phase one testing was conducted between 125 psi and 188 psi or 1.5 times its pressure class for two million cycles. A second phase was later added using the same pipe sample to evaluate occasional surges between 125 psi to 250 psi (two times pressure class) for 50,000 cycles. Currently, there are no known ASTM standards to evaluate large diameter HDPE performance under recurring surge pressures. This test complements other studies on the durability and reliability of large diameter PE4710 in water transmission systems.

The fatigue testing of a large diameter HDPE pipe was ranked with high priority during the WaterRF's project 4485 workshops (Najafi et al, 2015) with water utilities and other pipe professionals. The result of this test determines whether or not a 16-in. diameter HDPE (DR 17) can withstand cyclic loads that are 1.5 times its

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[1] The primary functions of the Hydrostatic Stress Board (HSB) of PPI are to issue recommendations to industry regarding the strength of thermoplastic piping materials intended for pressure applications, and to develop appropriate policies and procedures for the conduct of this activity. The HSB's recommendations are often referenced by North American plastics piping standards for the qualifying of thermoplastic piping materials for pressure piping service, and for the establishment of pipe pressure ratings.

pressure class for two million cycles. Two million cycles is equivalent to 100 years of service life based on 50 daily surges.

The HDPE pipe samples were delivered to CUIRE Laboratory on July 11, 2013. Table 1 presents pipe sample measurements. Figure 1 shows the pipe sample and the control sample.

Table 1. HDPE pipe sample measurements.

Pipe Number	Outside Diameter (in.)	Dimension Ratio (DR)	Pipe Wall Thickness (in.)	Pipe Length (ft)	Air Pressure Release Valve (in.)	Inlet/Outlet Tubes	
						Inner Diameter (in.)	Outer Diameter (in.)
Pipe Sample	16	17	0.94	14.97	¼	0.995	1.328
Control Sample	16	17	0.94	14.98	¼	0.996	1.325

Note: Dynamic Instantaneous Effective Modulus of HDPE Pipe, E_d = 150,000 psi

Figure 1. Pipe samples.

TESTING EQUIPMENT AND SETUP

This section describes the experimental setup and role of each device. The setup comprised of a 450-gallon water reservoir tank, a multi-stage centrifugal pump (10 HP), a data acquisition system, a control board, several pressure transducers, a DC power supply, one pipe sample (16 in. diameter), one control pipe sample, and control valves including one back-flow pressure valve, two solenoid/pressure ball valves, and two butterfly valves. Galvanized steel piping system with pipe diameters of 1-in. and 2-in. were used to connect water reservoir to the pipe sample. The PE4710 physical properties, such as modulus of elasticity and its viscoelastic nature were considered to design the test setup. Specifically, the PE4710 expansion and contraction and long-term surge pressure properties were used to calculate pump discharge, increase in water temperature, and the head-loss. Figure 2 illustrates a schematic diagram of the testing setup and equipment used. Table 2 presents a list of equipment provided for the project.

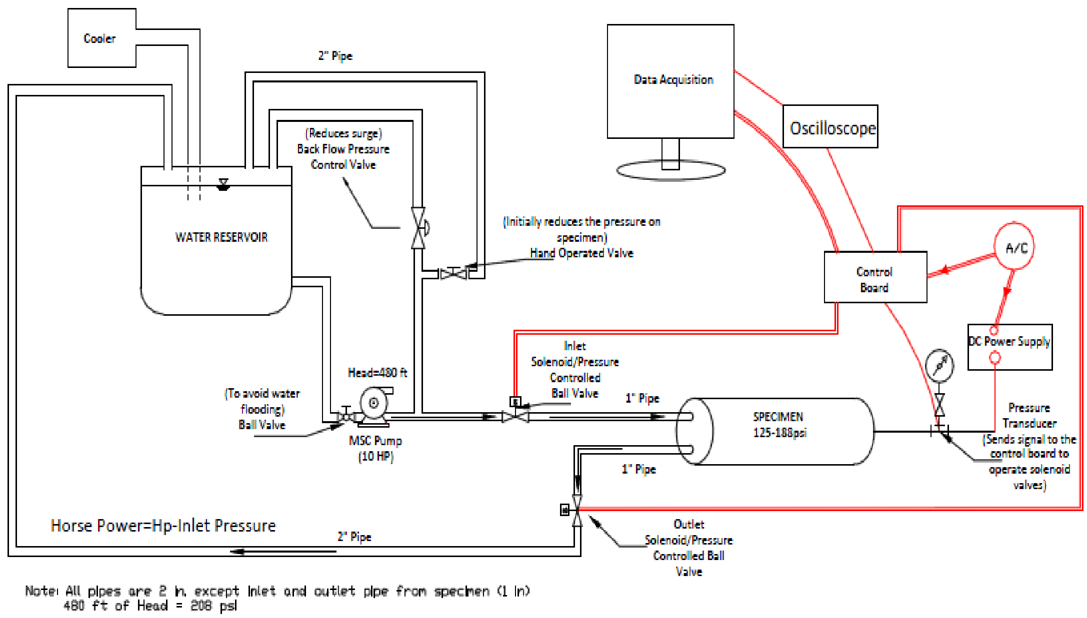


Figure 2. Schematic diagram of testing setup.

TESTING OPERATION[2]

Regular tap water was allowed to flow from reservoir to the pump which was located 10 ft (3 m) below bottom of the reservoir to create a head pressure of 480 ft. The pump delivered a pressure of 208 psi. Since the pressure cycles were between 125 psi and 188 psi; a "backflow control valve" was used to back pressure the extra water from the pump to reservoir, which was about 20 psi. The 188 psi pressure from the pump was used to pressurize the pipe sample using inlet and outlet solenoid valves. These valves were electrically operated using the control board (CB). One of the pressure transducers at the air release end of the pipe sample was connected directly to the control board. Once the water wave pressure activated this transducer, a signal was sent to the control board to operate solenoid valves. Another pressure transducer connected to the oscilloscope was used with data acquisition system to determine the waveform pattern.

Once the inlet valve opened, the pressure increased to 188 psi, and then the inlet valve was closed. The pressure impacted the pipe sample for approximately one second, and at this time, the outlet valve opened. Once pressure decreased to 125 psi, the outlet valve closed, and water from outlet valve went back to the reservoir. This process repeated for 2 million cycles.

[2] Although four pipe samples were delivered to the laboratory, due to time and budget constraints, the testing was performed on one 16-in. diameter 4710 HDPE pipe sample (AWWA C906).

Table 2. Equipment list.

Description	Quantity	Details
Multi Stage Centrifugal pump	1	10 HP
Back Flow Pressure Control Valve	1	Description: NPS 2 63EG Max Press: 285 psig
Solenoid/Pressure controlled Ball Valves	3	8210G027, 120/60, ASCO 1", 1" ORIF, 2NC, BR, GP, 225 PSI
Pressure Sensors (Transducers)	3	Model: PX209-200G5V, without LED display.
Water Reservoir	1	Diameter: 48"/Height: 5' Capacity: 450 gallons
Control Board	1	Part No. VPC 15055 FB107 consists of Isolated CPU, Touchpad LCD, Roclink 800 configuration, power supply.
DC power supply	1	24 V
Air Conditioning Units	2	-
Butterfly Valves	3	2 in.
Oscilloscope	1	PS2200A (PP906)

The control board was connected to a data logger to directly obtain the results from the data acquisition software. The oscilloscope was connected to the control board to determine the pressure wave from the transducer. To maintain the water pressure at 70°-73°F, two window air conditioning units were used with their grids inserted in the water reservoir, and water temperatures were held between 70°-73°F. Some factors influencing the testing conditions were:

1. Variation between maximum/minimum pressures.
2. Water temperature and room temperature.
3. Frequency and duration of surges.
4. Chemical substance present in the tap water.

PHASE 1 TEST RESULTS

In phase 1, the testing was performed for 2 million cycles. The pipe sample was periodically observed and measured for any dimensional changes. Figure 3 illustrates the cycle time of each surges (i.e., 8 to 12 seconds). The pressure cycle shows the cycle time of one complete surge.

Polyethylene is a viscoelastic material. Diameter of the pipe sample was observed to continuously increase over time due to impact of pressure surges. The diameter increase was mainly observed near the middle joint, but with no diameter changes at the end caps, because of their restraining effects. The 2-M cycles were completed in six months. At the higher temperature of 73°F, the cycle time increased to 12 seconds.

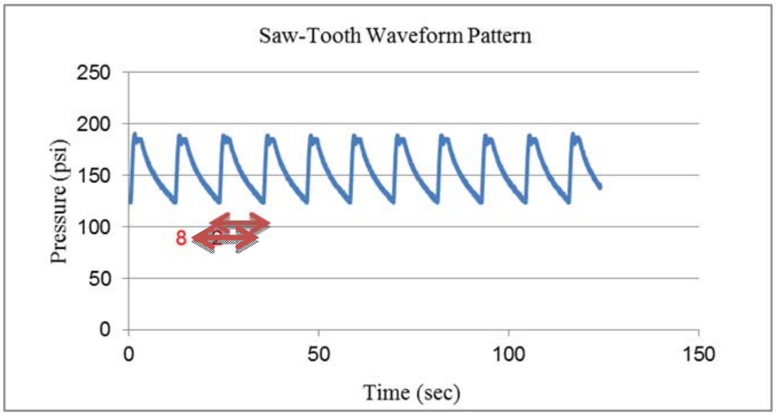


Figure 3. Saw-tooth waveform cycles.

Table 3 presents changes in the pipe diameter after 3 months. Compared to the control sample, the pipe diameter was increased by 0.27 in. Figure 4 illustrates the bulged pipe sample near the butt-fused joint. Figure 6 illustrates pipe sample measurement. At the conclusion of the testing (six months), there was 1-in. diameter increase.

Table 3. Diameter variations after 3 months.

Month	Diameter	Duration of cycle	No of cycles completed in millions
May 31st	16 in.	0 (start of test)	0
Sep 2nd	16.27 in.	8 sec	1.06

After three months (September 2nd), the pipe sample was no longer expanding along the length, and expansion of pipe sample started to stabilize along the diameter.

Figure 4. Pipe bulge near middle joint.

Figure 5 illustrates length measurement locations. Initially, no expansion along the length was observed. After three months, it was observed that as pipe sample diameter increased, length decreased. The length decrease continued until 1.52 million cycles, and after that the pipe length basically remained constant. Figure 6 illustrates the circumference measurement.

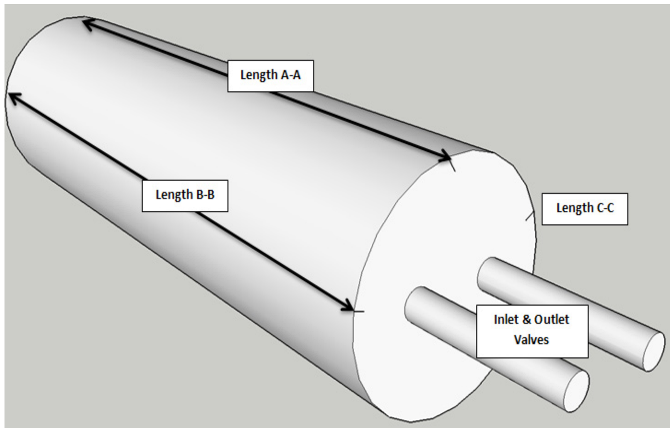


Figure 5. Length measurement locations.

Figure 6. Circumference measurement.

PE4710 Expected Life

To show how results of this testing can be used for estimating PE4710 design life; the following equation can be used to provide total number of surges for a 50- and 100-year design life (see Table 4).

Total No. of Surges = 50 surges/day x 365 days/year x Number of years.

Based on the following equations (Petroff 2013), and 50 pressure surges per day, Table 5 presents the peak stresses, cycles to failure, design life based on fatigue, and calculated pipe safety factor for a design life of 100 years.

$$\text{Number of Cycles} = 10^{\frac{1.708 - Log\left(\frac{Peak\ Stress}{145}\right)}{0.101}}$$

$$\text{Peak Stress} = (P_{PUMPING} + P_{SURGE}) * \frac{(DR-1)}{2}$$

Table 4. Number of surges for 50- and 100-year design life.

Years	No. of surges
50	912,500
100	1,825,000

Table 5. Cycles to failure for 16-in. diameter PE4710 .

Working plus surge pressure (WP + PS)	Peak stress (psi)	Cycles to failure	Fatigue life (years) @ 50 surges/day	Safety factor for 100 years @ 50 surges/day
1.2 x PC	1,246	45,907,200	2,515	25
1.5 x PC	1,504	7,123,000	390	4

Pipe Sample Dimensional Changes

The total difference between the initial and final diameter measurements was 0.52 in. After 1.76 million cycles, the diameter measurement did not change until 2 million cycles were reached. Table 6 presents expansion of pipe sample for one million and two million surge cycles.

PHASE 2 TEST RESULTS

The Phase 2 testing was conducted to evaluate resistance of HDPE pipe for occasional surge pressures up to two times its pressure class[3]. To perform this test, the research team had to replace the pump to a 15-HP pump, and the solenoid valves to 300 psi. For this test, the same pipe sample (with 2 M cycles completed in Phase 1) was used to pressurize from 125 psi to 250 psi for 50,000 cycles at 73° F. The test started on February 10, 2015, and ended on March 10, 2015. This test was not conducted continuously overnight, as it was done for Phase 1 testing. Figure 7 illustrates the saw tooth waveform cycles for Phase 2, with each cycle spanning 8 to 10 seconds.

[3] The 50,000 cycles for 100-years is approximately equivalent to 10 surges per week.

Table 6. Pipe sample diameter expansion

Surges	Expansion	
	in.	mm
1,000,000	0.27	6.858
2,000,000	0.52	13.21

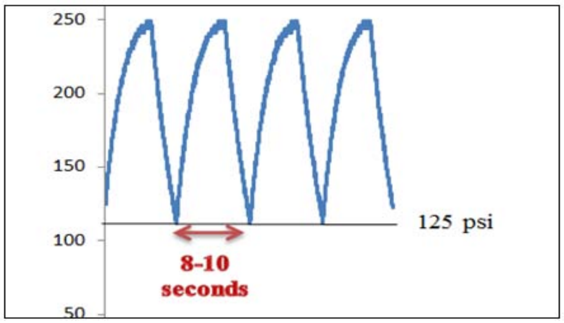


Figure 7. Saw-tooth waveform cycle for occasional surges.

Table 7 presents final diameter and length measurements after 50,000 occasional surges were completed.

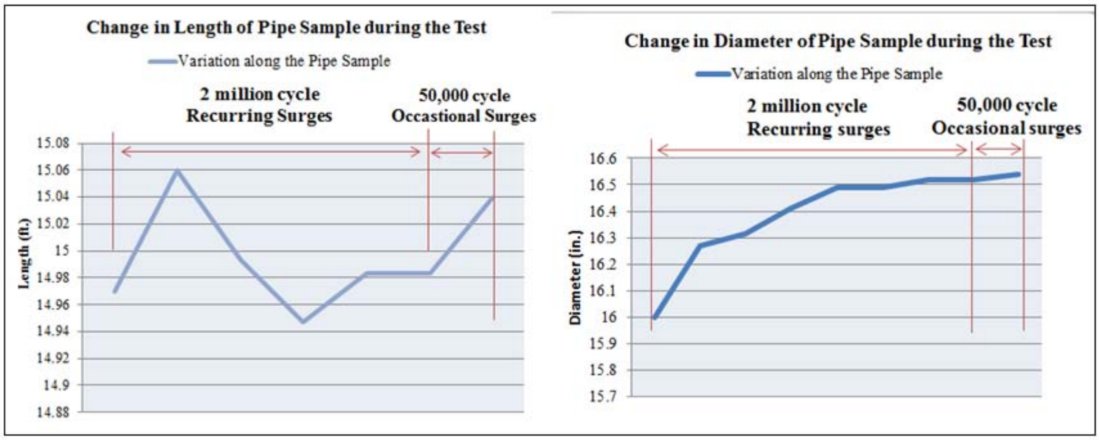
Table 7. Changes in pipe sample after 50,000 occasional surges.

Start date	End date	Diameter (in.)		Length (ft)	
		Before	After	Before	After
2/10/2015	3/10/2015	16.52	16.54	14.99	15.04

Figure 8 illustrates final length and diameter variations for both Phase 1 and Phase 2 of the project. The length measurements do not show good correlations with diameter measurements. This might be due to rounding issues during the measurements.

CONCLUSIONS

One area of durability that has not been thoroughly investigated is fatigue resistance to recurring and occasional pressure surges for large diameter HDPE pipes. This paper covered the experimental portion of WaterRF project #4485 to help in evaluating the reliability and durability of large diameter HDPE pipes. A testing methodology was developed and a 16-in., 15-ft, PE4710 pipe sample was tested for 2,000,000 cycles at 1.5 times pressure class. No failure was observed in the pipe sample, including the butt-fused joint, end caps, inlet and outlet tubes, and the air release valve. The same pipe sample was tested for an additional 50,000 cycles for twice pressure class and no failure was observed. The pipe sample dimensional variations were not uniform along the pipe due to stiffness of end seals and surge movement along the pipe length. The fatigue testing protocol developed in this project can be used to test other large diameter pipe materials.



(a) (b)

Figure 8. Variations in (a) length, and (b) diameter

ACKNOWLEDGEMENTS

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LIST OF ACRONYMS

ASTM – American Society for Testing and Materials
CUIRE – Center for Underground Infrastructure Research and Education
DR – Dimension Ratio (ratio of the pipe outside diameter to the pipe minimum wall thickness)
EPA – Environmental Protection Agency
HDPE – High Density Polyethylene
PVC – Polyvinyl Chloride
UTA – The University of Texas at Arlington
WaterRF – Water Research Foundation
WERF – Water Environment Research Foundation

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Reduce Diameter, Increase Capacity!

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Abstract

Providing adequate capacity to drain stormwater runoff during heavy rainfall events can be a challenge for many communities in Ohio and the Midwest. Even for the Municipal Separate Storm Sewer Systems (MS4) communities, flooding is common for rainfall events as short as two years. A number of methods are applied towards solving this problem, and they can be divided into three groups, which include gray infrastructure, green infrastructure (GI), or the combination of both. As an alternative method, trenchless pipeline rehabilitation by lining can provide a significant capacity increase to existing storm sewer systems by reducing the roughness of the storm sewer pipes. This study investigates the use of a semi-structural spray applied lining system to improve the hydraulics of a 42-inch (1,070 mm) stormwater pipeline that is comprised of reinforced concrete (RCP), brick, and corrugated metal (CMP) pipes.

Introduction

A number of methods have been applied over the years to prevent sanitary sewer and combined sewer overflows (SSO and CSO). These methods can be divided into three groups; i.e., gray infrastructure, green infrastructure (GI), or the combination of both. Gray infrastructure solutions include upsizing existing pipes using conventional (open-cut) or trenchless technology and piping/tunneling, whereas the most common GI techniques are comprised of Bioretention, Pervious Pavements, Rainwater Harvesting, and Structural Units.

As an alternative method, trenchless pipeline rehabilitation by lining can provide a significant capacity increase to existing storm sewer systems by reducing the roughness of the storm sewer pipes. Pipe lining is a common rehabilitation method and mostly applied to sanitary sewers to prevent groundwater/rainwater entry. It also improves structural integrity, as well as the service life and economic value of a sewer system. Nevertheless, rehabilitation is typically perceived as a method that decreases the hydraulic capacity of a pipeline due to reduction in the cross-section area. While it is true that lining a pipeline can significantly reduce the inside diameter (ID) of the pipe, the change in hydraulic capacity is also directly affected by the surface roughness (Manning) coefficient, which could vary from 0.009 for very smooth interior to higher than 0.030 for channels with protrusions.

This study investigates the use of a semi-structural spray applied lining system to improve the hydraulics of a 42-inch (1,070 mm) stormwater pipeline that is comprised of reinforced concrete (RCP), brick, and corrugated metal (CMP) pipes. It's based on a storm sewer improvement project in Delaware, Ohio. The project area (Bernard Avenue) is a linear site along a roadway bounded by the railroad adjacent to Toledo Street on the west, Park Avenue to the north, Bernard Avenue to the south, and the Olentangy River to the east.

A hydraulic and hydrologic (H&H) analysis was carried out by the project team to verify runoff from the existing watershed and compare that to the capacity of the existing drainage system. The existing runoff was calculated using the City of Delaware's 2013 Infrastructure Design Guide (IDG). The H&H analysis results confirm the field observations of flooding in discrete areas throughout the project area. Upon further analysis and field investigations, the project team concluded that lining the downstream part of the pipeline, along with other measures, will result in a significant increase in the stormwater discharge capacity, thereby keeping the hydraulic grade line (HGL) below surface beyond a 10-year storm event.

Bernard Avenue Project

Bernard Avenue is an east-west road located just south of downtown Delaware, Ohio (City). It parallels the main drainage pathway in this small watershed within the City, and has become synonymous with the stormwater drainage basin. The Bernard Avenue stormwater drainage basin is made up of a combination of residential, commercial, park, and institutional land uses. The basin has experienced flooding problems for the past couple of decades, which continue today. The City has been working on stormwater improvements based on prioritizations from previous studies (Burgess and Niple, 2012, City of Delaware, 1990), and the Bernard Avenue basin has become the next target area for improvements. Delaware requested proposals from professional engineering consultants to perform this work and selected American Structurepoint (Engineer) to help determine the best solution for drainage improvements in the Bernard Avenue basin (Figure 1).

The Bernard Avenue basin slopes west to east from the CSX railroad tracks to the Olentangy River. The approximate boundary of the drainage basin is Toledo Street/CSX railroad to the west, Park Avenue to the north, Bernard Avenue to the south, and the Olentangy River to the east. The western and central portions of the basin include residential, commercial, and park land use. The eastern portion of the basin is almost completely taken up with Ohio Wesleyan University's campus. The total basin area is approximately 125 acres (50.6 hectares).

The Bernard Avenue basin has had a drainage system in some form since this part of the City was developed. The backbone of the drainage system is a stone and mortar channel, which follows part of the original drainage pathway through the basin. In the intervening years, this main channel has been covered over and additional storm sewers were installed. In some cases buildings were actually constructed over the channel and storm sewers. In most cases, the channel and sewers are old enough that Delaware does not have documented easements over the drainage infrastructure. Although Delaware continues to maintain the system, the lack of clear access has the potential to cause problems. Perhaps the most significant problem in the basin is the documented flooding. The flooding has been especially bad between Liberty Avenue and Sandusky Street, which make up the central third of the basin, although some has occurred upstream of Liberty Avenue as well.

With the above issues, Delaware has determined a need to address the aging infrastructure, increase capacity, and improve accessibility in the Bernard Avenue basin. The scope of the Bernard Avenue project includes review of existing information, preparation of a hydraulic and hydrologic analysis, selection of a preferred alternative, design, easement preparation, and construction plan development.

Methods Evaluated

One of the first tasks the Engineer completed was evaluating several different alternatives to improve the stormwater system in the Bernard Avenue basin. The following alternatives were considered:

1. Installing new stormwater trunk sewer and inlets, sized accordingly for the drainage area and level of service
2. Rehabilitating the existing sewers that are structurally sound using trenchless methods
3. Regrading rear yards to provide more effective runoff to stormwater infrastructure
4. Creating a hybrid ditch consisting of a sand/soil matrix over a perforated drain pipe
5. Constructing a combination of green infrastructure and off-line detention in key locations along the drainage corridor

Each of the above alternatives was evaluated qualitatively and in some cases quantitatively to determine the best application for addressing drainage problems in the basin. The first step before any of these alternatives could be evaluated was to complete a hydraulic and hydrologic analysis of the Bernard Avenue basin.

Hydraulic and Hydrologic Analysis

The hydraulic and hydrologic (H&H) analysis for the Bernard Avenue stormwater drainage basin was completed using a combination of spreadsheets and modeling software (XPSWMM). The basin is small enough that Engineer and Delaware agreed the runoff analysis would be done using the *Rational Method*. This method traditionally returns more conservative flows which in turn provide a better safety factor in sizing improvements.

Figure 1. Bernard Avenue project area and watershed boundary.

Updated contours from the local GIS database were used to check actual drainage boundaries for the basin. Some minor modifications were made from previous studies. The Bernard Avenue basin was then broken down into 17 sub-basins. Delaware has developed their own runoff coefficients based on the type of soil and overall permeability throughout the ground surface. These were used to determine runoff coefficients for each sub-basin; and then, time of concentration was calculated for each sub-basin. The longest flow path was determined for the entire basin and peak flow rates were determined. Three recurrence intervals were considered; i.e., 2year, 5-year, and 10-year. The flow rates for the three recurrence intervals were then input into the model to get the overall flow rate and check for flooding locations.

Model Parameters/Assumptions

A hydraulic and hydrologic (H&H) model of the existing stormwater system was run, and it was used to analyze the proposed improvements. The goal of using the model was to optimize the selected stormwater improvements while still achieving the City's requirements. Delaware requires new stormwater improvements to keep the hydraulic grade line within the pipe for 2-year events and below the top of structure for 5-year events. Delaware also preferred the 10-year event remain below the top of structure. As such the model scenarios included 2-year, 5-year, and 10-year/24-hour storm events for each the existing system and each improvement considered.

When the model was built, assumptions were made for the friction coefficient of each existing sewer segment based on the records obtained via a closed circuit television (CCTV) of the conduits. This part of Delaware's stormwater system had not been updated since original installation, which was believed to be more than 100 years ago for some (stone and mortar) pipe segment. Pipe material changed from concrete to stone to clay and back. The age of the system and deterioration of the materials created a situation in which the friction factors (Manning's n) were high. These were estimated to range from 0.013 up to 0.030 for the stone and mortar channel. Qualitative evidence observed by City personnel indicated that the model results of the existing system were reasonable.

The model was executed only for the enclosed conveyance portion of the basin. This started at Liberty Street and continued through Ohio Wesleyan University's campus east to the Olentangy River. The existing drainage system in this portion of the basin could be described in four distinct sections. The first section is smaller diameter, circular pipe (24-inch/610 mm to 27-inch/690 mm) from Liberty to Franklin. The second section is the stone and mortar channel from Franklin to east of Sandusky. The third section goes back to a larger, circular pipe (42-inch/1,070 mm) through Ohio Wesleyan University's campus from east of Sandusky to Henry. It is the steepest section of the drainage system and runs through the middle of the Ohio

Wesleyan Campus. The final section is a circular pipe (42-inch/1,070 mm) on a flatter average slope (1.3 percent) from Henry out to the Olentangy River.

Two key areas were revealed during the modeling. The stone and mortar channel, while supporting a large cross-sectional area, was creating a capacity restriction due to its high friction coefficient and relatively flat slope. The second area was the last pipe section from Henry Street to the Olentangy River. It also had a relatively high friction factor, and was also the flattest slope throughout the system. The model revealed that water surcharged out of the system mainly in three locations: upstream end of pipe where it collects flow from an open ditch, manhole between Washington and Franklin Streets, and a manhole upstream of US 23. The surcharge locations are represented by blue waves in Figures 3 and 4.

Results

A total of seven (7) improvements were analyzed in the model. Alternatives 1-5 were different alignments for replacement of stormwater conveyance from Liberty to east of Sandusky and rehabilitation of the pipe from Sandusky to the Olentangy River. Alternatives 1-4 were eventually dropped in favor of Alternative 5. Three variations of Alternative 5 were then analyzed to determine the optimal length of sewer to rehabilitate. The goal was to rehabilitate enough pipe segments to eliminate surcharging while not spending money to rehabilitate pipes, which will not reduce surcharges.

The optimized solution was parallel sewer installation from Liberty to Sandusky and rehabilitation of the pipe segment downstream from Henry to the Olentangy River (Figure 2).

Sewer Rehabilitation in the Downstream

To mitigate overflows for a 5-year/24 hour storm event, the current design includes installation of a parallel sewer line between Liberty Street and Franklin Street, ditch improvements, green infrastructure at the Ohio Wesleyan University Campus, and lining of the downstream pipe segment (from Henry Street to discharge point at US 23) to improve hydraulic capacity. The remainder of this paper will focus on the proposed hydraulic capacity improvement by lining the downstream end using semi-structural (epoxy) lining system.

Manning's equation is widely accepted as a method of calculating flow capacity in an open channel flow such as gravity sewer pipes:

$$Q = A\frac{1}{n}R^{2/3}S^{1/2}$$

Where Q is the flow rate, A is the flow cross-sectional area, n is the roughness coefficient, R is the hydraulic radius, and S is the slope of the channel (pipeline).

Manning's equation suggests the flow capacity of a gravity flow pipe is inversely proportional with the roughness (Manning) coefficient, n.

Often times sewer rehabilitation by lining is regarded as a solution that would reduce hydraulic capacity; nevertheless, any change to the hydraulic capacity due to lining is dependent on the following factors:

1. Roughness of the host pipe
2. Roughness of the lining
3. Thickness of the lining
4. Benchmark used for comparison (i.e., existing vs. design capacity).

Table 1 indicates the values used for the analysis of the downstream hydraulics in the Bernard Avenue basin.

Table 1. Downstream pipeline parameters associated with system hydraulics.

Parameter	**Justification for Value Used**
Roughness of Host Pipe	From 0.015 for concrete pipe to 0.030 for the cobble stone/mortar conduit. Concrete pipe has been in service for an extended period of time
Roughness of Lining	0.011 used as a mean value, factoring in the protrusions along the host pipe
Thickness of Lining	8 mm average (300 mils)
Comparison Benchmark	Existing condition used for the H&H model. Assumptions of the n value based on literature and past experience of the authors

Based on the foregoing parameters and assumptions, flow capacities for the lined and unlined pipes were calculated based on Manning's Equation. The results (see Figure 5) suggest using a semi-structural liner can improve the hydraulic capacity of the 42-inch (1,070 mm) pipeline up to 40 percent.

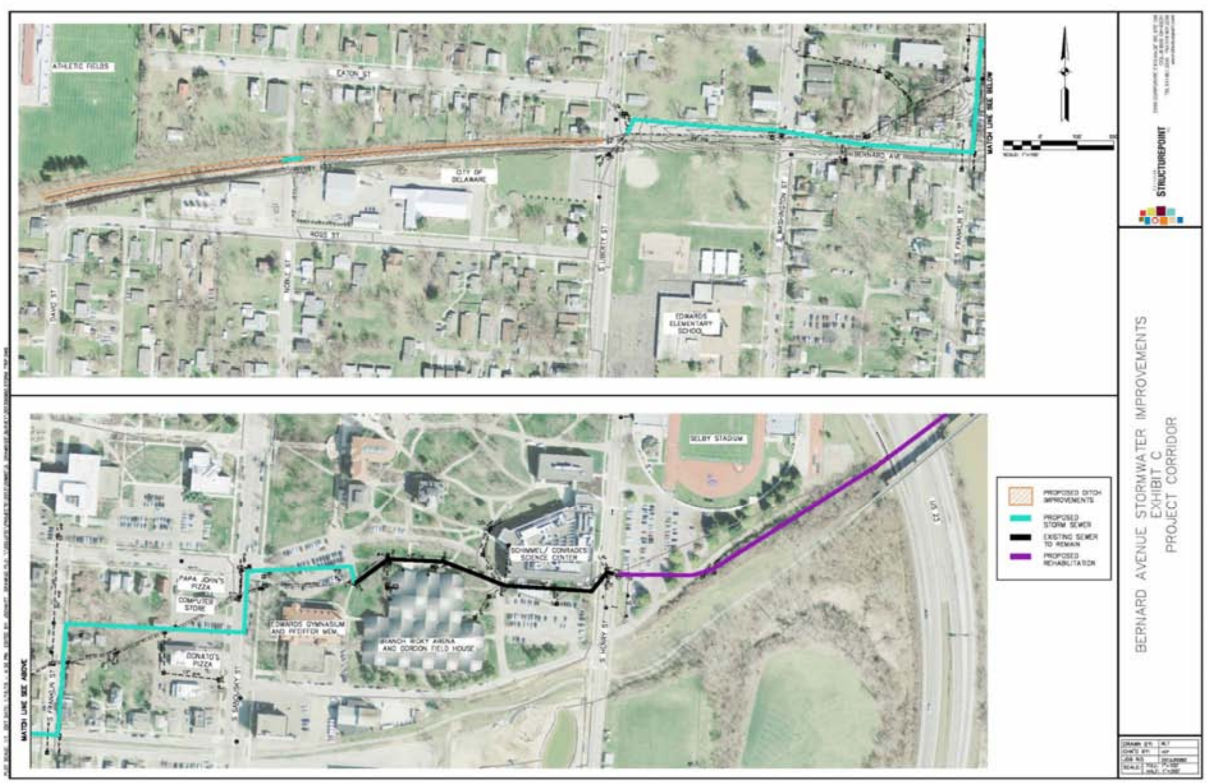

Figure 2. Planview of the proposed improvement on aerial.

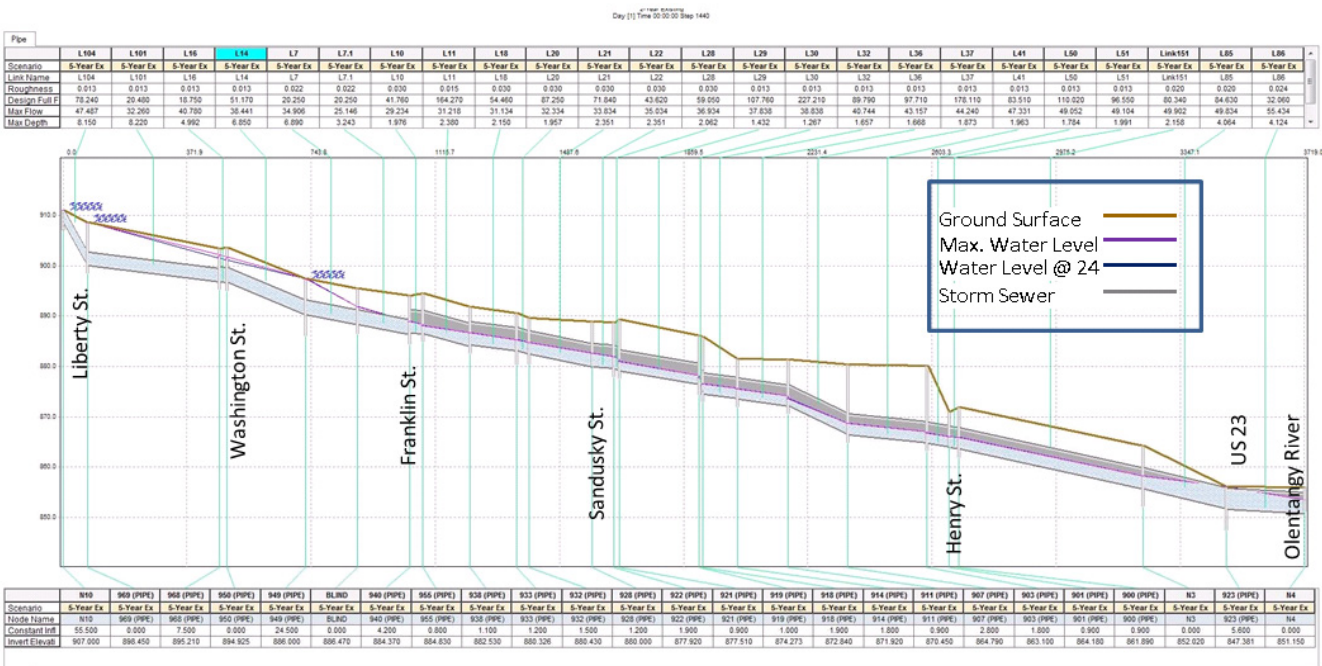

Figure 3. Hydraulic grade line for 2-year storm event on the existing system. The wave symbol indicates overflow. (Refer to Figure 1 for node numbers.)

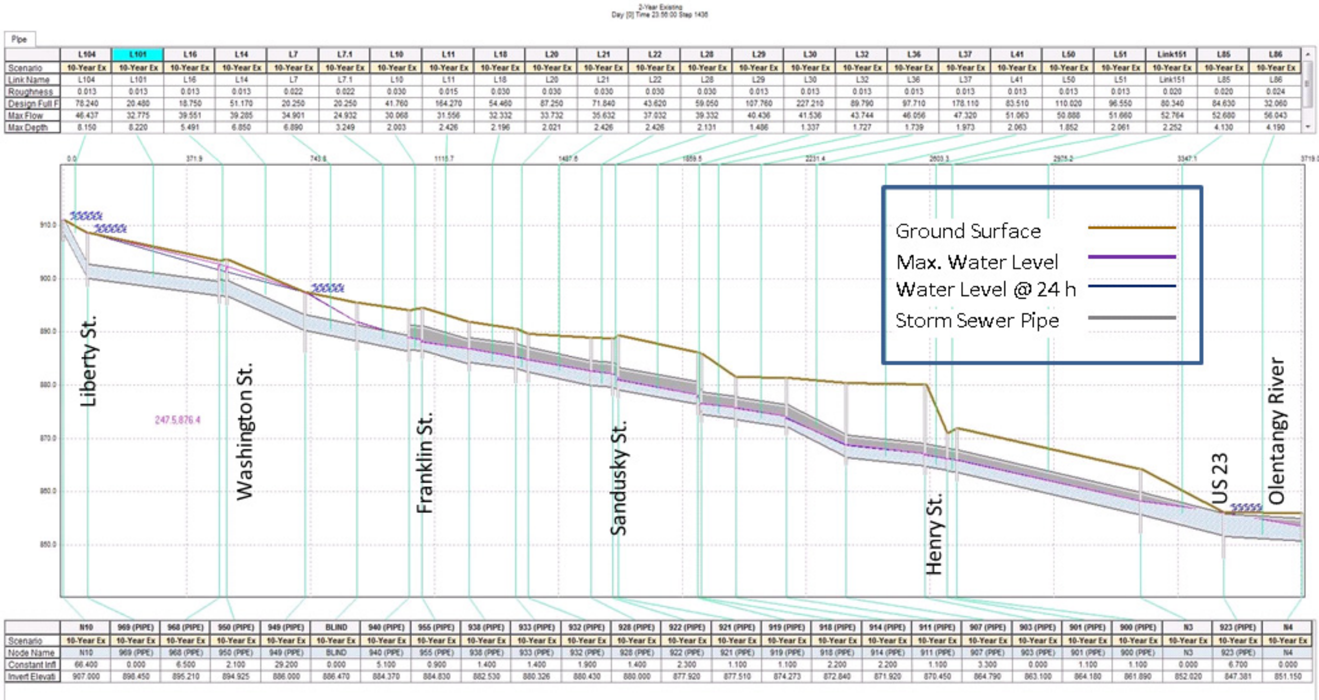

Figure 4. Hydraulic grade line for 10-year storm event on the existing system. The wave symbol indicates overflow. (Refer to Figure 1 for node numbers.)

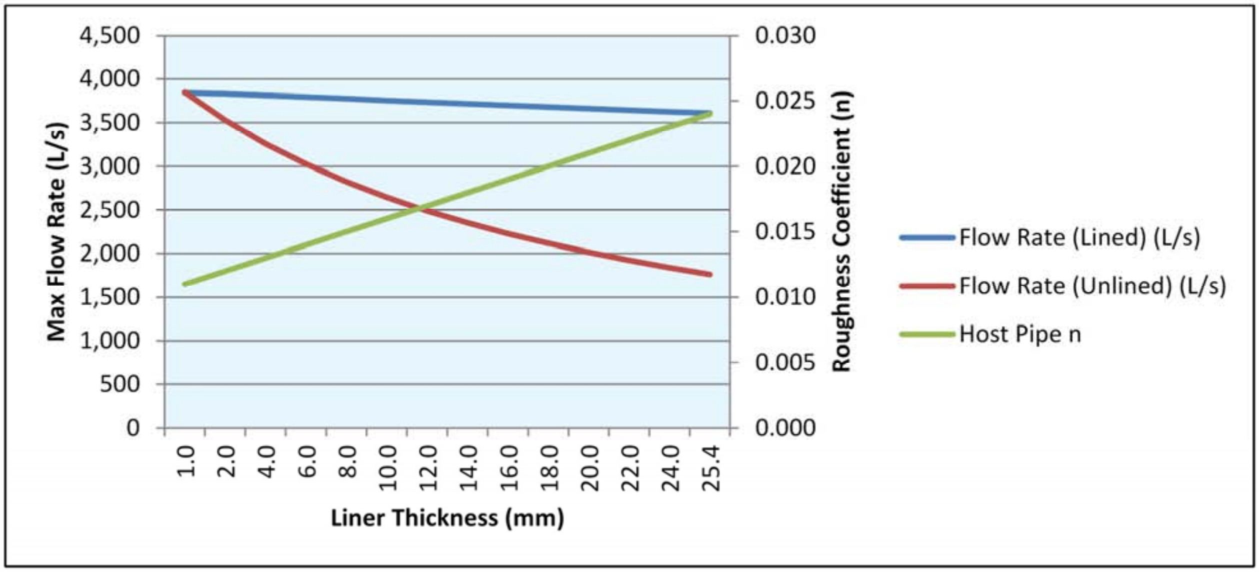

Figure 5. Pipe flow capacity versus liner thickness for a range of roughness coefficients.

Concluding Remarks and Future Direction

The hydraulic analysis performed herein as a part of the design efforts for Bernard Avenue Stormwater Improvements suggest:

1. Lining a gravity flow system can significantly improve the hydraulic capacity, and might enable meeting the target level of service without the need for upsizing or parallel sewer installation. This is dependent on essentially four factors outlined in this technical paper:
    a. Roughness of the host pipe
    b. Roughness of the lining
    c. Thickness of the lining
    d. Benchmark used for comparison (i.e., existing vs. design capacity).

2. Even a thick (one inch or 25 mm for this case), fully structural liner can improve the hydraulic capacity, and this is dependent on the factors (a) and (d) listed above.

3. The analysis provided herein assumes approximate values for the roughness (Manning) coefficient for the unlined and lined pipe. While this is deemed adequate by the project team to move forward towards final design and installation, determining the actual *n* value of the host pipe prior to selecting a renewal option is recommended for larger scale projects.

4. More research is needed to determine the effects of protrusions on the mean roughness coefficient of a pipeline lined with spray applied semi-structural liner.

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Acknowledgment

This technical paper is based on a stormwater improvement project for the City of Delaware, Ohio. Authors would like to extend their gratitude to Mr. Brad Stanton, Public Works Director, for his support for the project and presentation of this paper.

How to Estimate Flow Area Reduction and Excessive Roughness Effects in Aged Pipelines

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Abstract

Accurate hydraulic analysis and head loss calculations are essential for conveyance pipeline design. Flow modeling tools significantly expedite the hydraulic analysis steps, but estimating nominal pipe characteristics may result in considerably erroneous results. This paper presents critical design requirements to estimate flow area reduction and quantify excessive roughness effects in aged pipelines. Correction factors shall be applied to the Hazen-Williams equation for turbulent flow to reflect the actual flow area and corrected roughness coefficients. Two scenarios of an actual design will be presented to illustrate the order of magnitude of erroneous results when the design is based solely on nominal flow area and basic roughness factors. In addition to defining extreme conditions, this paper describes experimental methods, boundary conditions, and field measurements required to develop hydraulic characteristics specific to aged systems. This paper also presents general corrective actions required to improve hydraulic performance of existing systems.

DESIGN FUNDAMENTALS AND OBJECTIVES

For systems with high dynamic losses (e.g. long length), analysis and modeling should be based on the actual pipe inner diameter and roughness. For example, the Hazen-Williams equation is a popular method for calculating friction losses using either hand calculations or hydraulic model developments. Traditional equations (Streeter 1985) define "d" as pipe diameter without correction factors for actual conditions. The Hazen-Williams equation for turbulent flow is:

$$h_{f\,(ft)} = \left(10.44\ L_{(ft)}\ Q_{(gpm)}^{1.85}\right) / \left(C^{1.85}\ d_{(in)}^{4.87}\right)$$

Where:
h_f: Head loss in feet of water
L: Length of pipe in feet
Q: Flow rate in U.S. gallons per minute (gpm)
C: Hazen-Williams roughness coefficient
d: Inner diameter (ID) of the pipe in inches

The simplicity of modeling tools used for design may undermine the importance of meaningful and correct input data. For example, as shown in Figure 1, the default programmed value of inner diameter for a 36-inch ductile iron pipe is 37.34 inches.

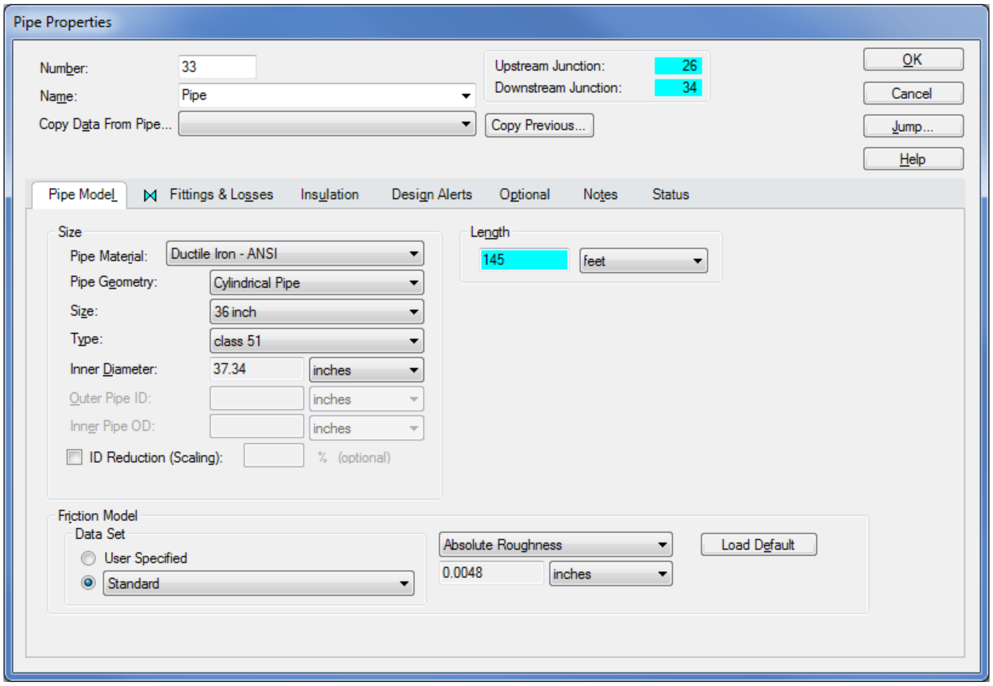

Figure 1. Example modeling program pipe properties input.

The hydraulic characteristics of a conveyance system change as it is exposed to corrosive fluid and settleable solids. The cross section of the conduit will be reduced due to solids deposition. Also, corrosion will increase surface roughness of the conduit. Correction factors should be applied to the input values for the Hazen-Williams equation to reflect the actual flow area and corrected roughness coefficients. Note that double lining of pipes is a popular method to protect pipes in corrosive environments and additional lining will reduce pipe ID and conveyance capacity. Assuming nominal pipe characteristics may result in considerably erroneous results.

Roughness coefficient (C) values of 140 to 150 are often chosen for initial analysis of new water piping (Lindeburg 2006). Usually, C values of 90 to 100 are acceptable estimates for aged pipelines. C values of 60 to 80 can be used for aged pipes with significantly rough condition. After factoring applicable C values and inner diameter reduction in design calculations, the calculated head loss would be considerably higher for aged conveyance systems. For example, only 25 percent ID reduction and C value of 90 in pipelines conveying suspended solids (e.g. raw wastewater influent or river water) will result in dynamic losses of approximately ten times higher than the initial (new) condition. Assume $\mathbf{C}_{new} = 147$, $\mathbf{C}_{old} = 90$, and $(\mathbf{d}_{old} / \mathbf{d}_{new}) = 0.75$:

$$\rightarrow \ (\boldsymbol{hf}_{old} \ / \ \boldsymbol{hf}_{new}) = (\mathbf{C}_{new}^{\mathbf{1.85}}\ \mathbf{d}_{new}^{\mathbf{4.87}} \ / \ \mathbf{C}_{old}^{\mathbf{1.85}}\ \mathbf{d}_{old}^{\mathbf{4.87}}) = 10$$

Note that Hazen-Williams roughness coefficient, C, value of 90 and 25 percent ID reduction are relatively common and realistic conditions for aged systems conveying suspended solids.

DESIGN EXAMPLE

Project Description – Pre-Design Condition:
Raw sewage influent was lifted to the plant processes by two existing pump stations. The designer's initial assessments recommended replacement of the pump stations with a single new influent pump station. The project preliminary design report identified design flows for the new influent pump station for a 3-phase improvement plan. The existing conveyance pipeline was required to remain in service during design and was therefore not accessible, so designers were unable to evaluate the actual condition and hydraulic characteristics of the existing conveyance system. The preliminary design report recommended utilizing the existing conveyance system and connecting the new pump station common discharge to the existing 36-inch pipeline.

Project Planning Criteria:
The design flow requirements were previously developed during the project initial assessment. A summary of these flow rates are listed in Table 1.

Table 1. Design Criteria

Estimated Parameter	Phase I	Phase II	Phase III
Peak Hour Flow (mgd) [a]	45	50	52
Average Diurnal Low Flow (mgd)	6	8.5	9
Number of Pumps[b]	5	6	6
Peak Flow Required per Pump (gpm)[c]	7,500	7,000	7,200

[a] million gallons per day (mgd).
[b] Number of pumps includes one redundant (standby) unit.
[c] gallons per minute (gpm).

Project Final Design:
Considering that the two existing pump stations and associated piping would be demolished and replaced with new piping, conventional field pumping tests using the existing pressure gauges at the pump discharge piping would not provide an accurate determination of the existing system head losses. During final design, the designer estimated a Hazen-Williams roughness coefficient of 130 for the existing 36-inch welded steel discharge pipeline. Also, the nominal ID of 36 inches was assumed for the entire discharge pipeline. The new influent pump station has been designed and constructed. The station has a total of five initial variable speed influent pumps to convey the screened influent through the existing 36-inch discharge pipeline to the existing grit facility.

Hydraulic Modeling:
Figure 2 presents the hydraulic model developed during final design using AFT Fathom®.

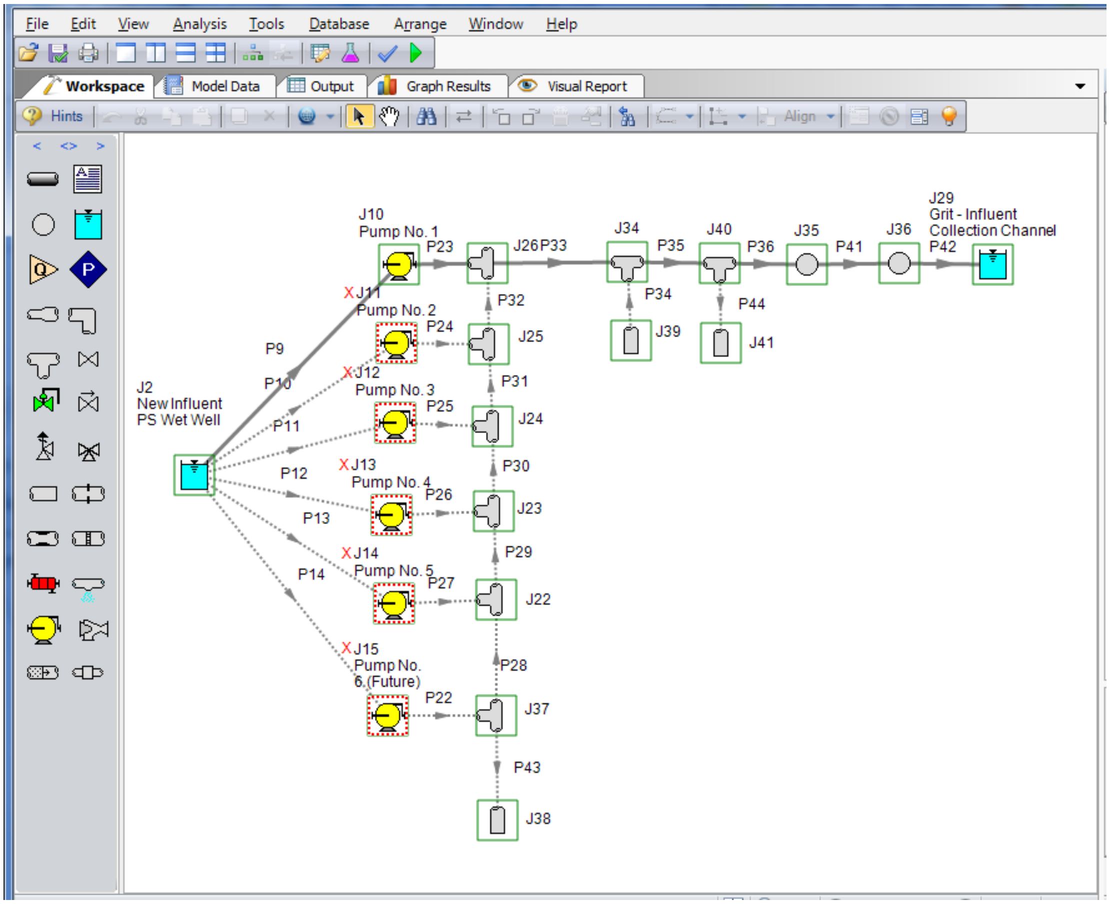

Figure 2. System flow model.

Project Start-up and Field Testing:
During start-up and testing, field test reports using two pumps indicated that the actual system head loss was higher than estimated values used in final design. Higher actual system head resulted in a lower total flow with four pumps running than the initial design capacity, by about 7,000 gpm. A summary of field test results are listed in Table 2 and shown in Figure 3.

Table 2. Test Results

Operating Condition	Anticipated TDH [a] (ft)	Actual TDH (ft)	Capacity Reduction (gpm)	Capacity Reduction (%)
One (1) Pump at Full Speed	44	48	250	2.5
Two (2) Pumps at Full Speed	48	56	2,500	13
Three (3) Pumps at Full Speed [b]	55	63	4,500	17
Four (4) Pumps at Full Speed [b]	60	68	7,000	23

[a] Total Dynamic Head (TDH).
[b] Extrapolated from field test results for two operating pumps.

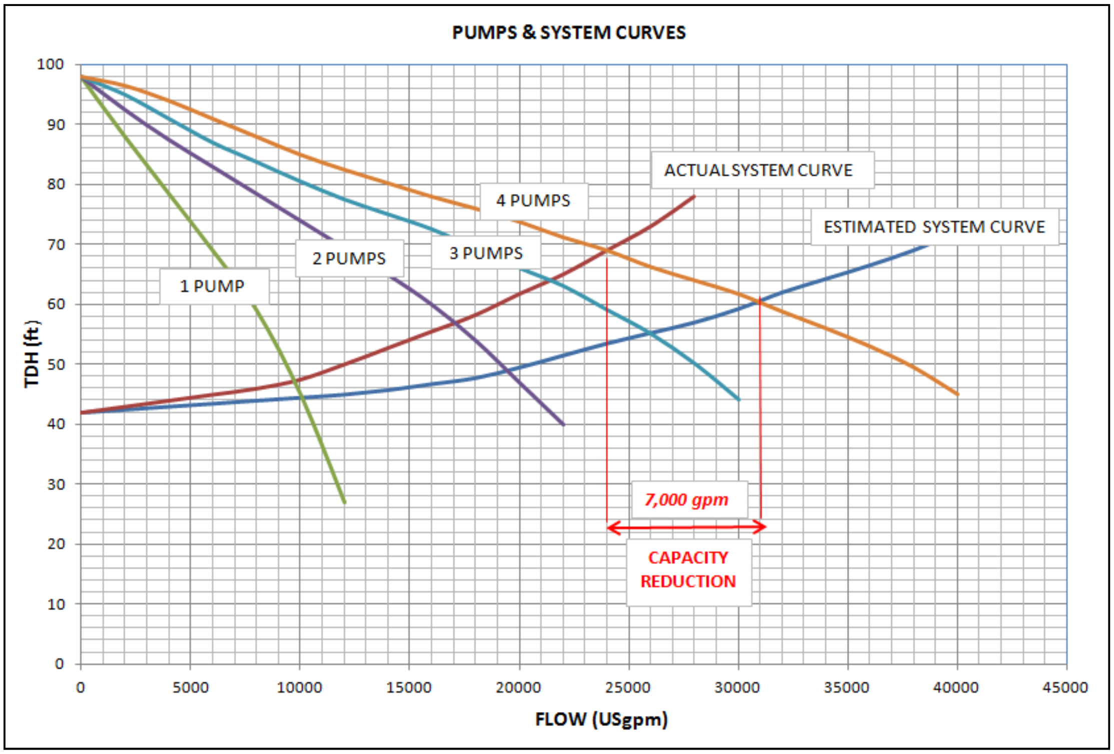

Figure 3. Estimated system curve vs. actual system curve.

After startup a field investigation was completed to identify the primary reasons for the additional system head loss. It was determined that actual roughness coefficients of 110 to 115 were the practical values for the existing pipeline. The estimated coefficient range was calculated based on approximately 10 to 12 percent ID reduction. The ID reduction is attributed to very low flow occasions during initial operation of the existing 36-inch force main; less than 1 feet-per-second (fps) flow velocity. The design team proposed a number of solutions for potential corrective actions.

CORRECTIVE ACTIONS

Field investigation:
One of the most effective methods of field investigation is to assess the system condition and look for deficiencies which affect hydraulic performance. Primary deficiencies are often one or more of the following:

- Partially closed valve during system hydraulic testing
- Partially closed valve due to valve stem failure
- Unaccounted for fittings (such as reducers)
- Downsized segments upstream and downstream of flow meters
- Clogged segments
- Grease accumulation
- Oval cross sectional area due to pipe deflection
- Inaccurate flow meters or pressure gauges

Pipeline Cleaning:
If the downstream process is capable of handling higher than designed flow rates, high velocity flow of 8 to 10 fps are recommended for flushing the network for an extended period of time and then re-evaluating the system capacity. Mechanical cleaning (pigging) is another viable cleaning option.

Flow Capacity Increase:
Changing the pump impellers or adding more pumps can provide additional capacity, but require higher power draw. Installing a parallel pipeline for the most inefficient segments might also be considered as a last resort to provide additional capacity.

DESIGN CONSIDERATIONS

New Pipe - Anticipate future service conditions:
If a new pipeline replaces an existing system, the design engineer must define future service conditions. The design should anticipate future hydraulic characteristics and propose mitigation measures. Future reduction in flow area and change in roughness should be estimated to quantify capacity reduction in the future. Also, for systems with pumps, the aged impeller will lose the initial rated flow (de-rated) which negatively affects the system flow capacity.

Quantifying capacity reduction for the future services is the pre-requisite of designing a system which mitigates for future losses. For instance, the designer may estimate an average reduction in the C value (e.g. 1%) and pipe ID (e.g. 1%) every year of service life. The average annual reduction in the C value and pipe ID should be estimated based on field investigation. Existing pipelines should be evaluated to estimate future operating conditions of a new system.

If existing pipelines are not accessible, the designer may estimate the average annual reduction in the C value and pipe ID based on condition of similar installations. However, both service and pipe material should be the same to develop an accurate estimate.

Based on the significance of capacity reduction, the designer may consider mitigation measures such as dividing future flow between a larger number of smaller constant speed pumps, phased pump installation, variable speed flow control, or even planning for a parallel pipeline installation in the future. Both design assumptions and mitigation measures would be project specific. Unique flow characteristics of each system should be considered to develop project specific mitigation measures.

Aged Pipe - Define design criteria:
If the existing system will be utilized, the most practical approach to estimating actual head losses is to define the design criteria by experimental methods.

This paper proposes an experimental process shown in Figure 4 for existing systems.

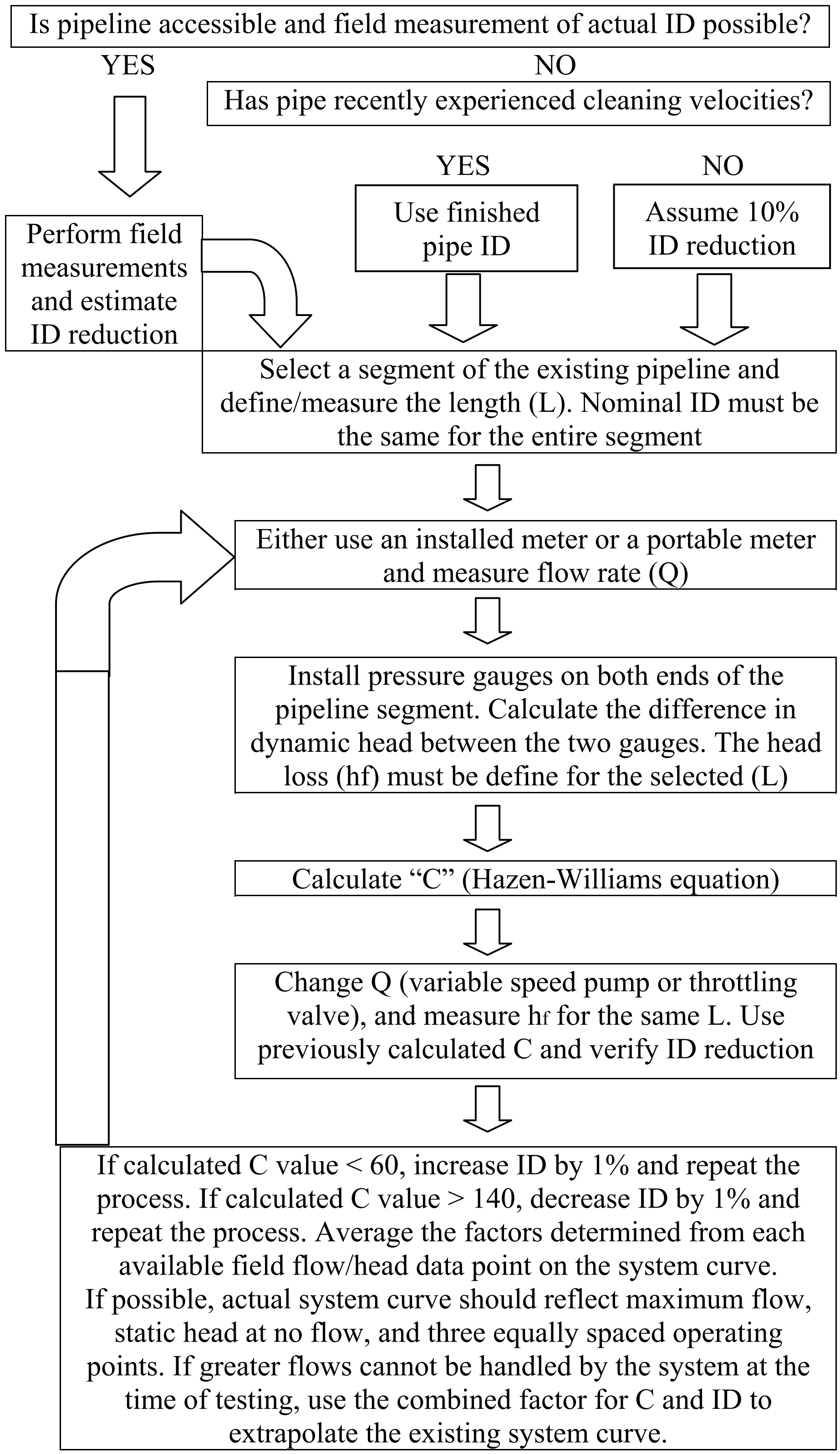

Figure 4. Experimental process flow diagram.

The process determines the value of a combined factor for actual C and ID, which is necessary to characterize the actual hydraulic conditions for the existing system. Exact values for the actual ID of the pipeline can only be determined through physical examination and measurement. However, it is sufficient to know the combination of the actual C and ID values. If necessary, this experimental process can be performed for each unique segment of the pipeline.

The equation for the combined factor for existing conditions is:

$$\Rightarrow (\mathbf{C}_{\text{actual}}^{\mathbf{1.85}}\, \mathbf{d}_{\text{actual}}^{\mathbf{4.87}}) = 10.44\, \mathbf{L}_{(\text{ft})}\, \mathbf{Q}_{(\text{gpm}),\ \text{test}}^{\mathbf{1.85}} / (\boldsymbol{h}_{\mathbf{f},\ \text{test}})$$

The combined factor for existing pipe conditions can be used to describe the hydraulic characteristics for each segment of the system considered, and the factor can be used to extrapolate system performance for flows greater than the test flows if higher test flows cannot be accommodated during the time of testing.

Define Boundary Conditions:

Knowing the worst case condition will enable the design engineer to define conservative design factors like required power, pipe acceptable ID range, estimated cost, etc. Again, the Hazen-Williams equation can be utilized to define the boundary conditions for each pipe size. So, how bad is the existing pipe?

For old piping with high solids solution, one option is to assume C value of 90 and ID reduction of 25 percent as the upper (high head) boundary condition. The proposed option defines the best first trial for a trial-error-process and probably the worst case scenario.

Also, C value of 150 and no ID reduction represents the lower (low head) boundary condition. Using the Hazen-Williams equation:

For upper boundary conditions; $\mathbf{L} = 100$ ft, $\mathbf{C}_{\text{old}} = 90$, and $\mathbf{d}_{\text{old}} / \mathbf{d}_{\text{new}} = 0.75$;

$$\Rightarrow \boldsymbol{h}_{\mathbf{f}\,(100\ \text{ft})} = \mathbf{0.253Q}_{(\text{gpm})}^{\mathbf{1.85}} / (\mathbf{0.75d}_{\mathbf{nominal}\,(\text{in})})^{\mathbf{4.87}}$$

For lower boundary conditions; $\mathbf{L} = 100$ ft, $\mathbf{C}_{\text{new}} = 150$, and $\mathbf{d}_{\text{old}} / \mathbf{d}_{\text{new}} = 1.0$;

$$\Rightarrow \boldsymbol{h}_{\mathbf{f}\,(100\ \text{ft})} = \mathbf{0.098Q}_{(\text{gpm})}^{\mathbf{1.85}} / \mathbf{d}_{\mathbf{nominal}\,(\text{in})}^{\mathbf{4.87}}$$

Figure 5 shows upper and lower boundary conditions for the design example previously described. As shown in Figure 5, the actual system curve is almost in the middle of the boundary conditions. As the pipeline ages, the actual system curve moves toward the upper boundary. Corrective actions like pipeline flushing or cleaning would move the actual system curve toward the lower boundary.

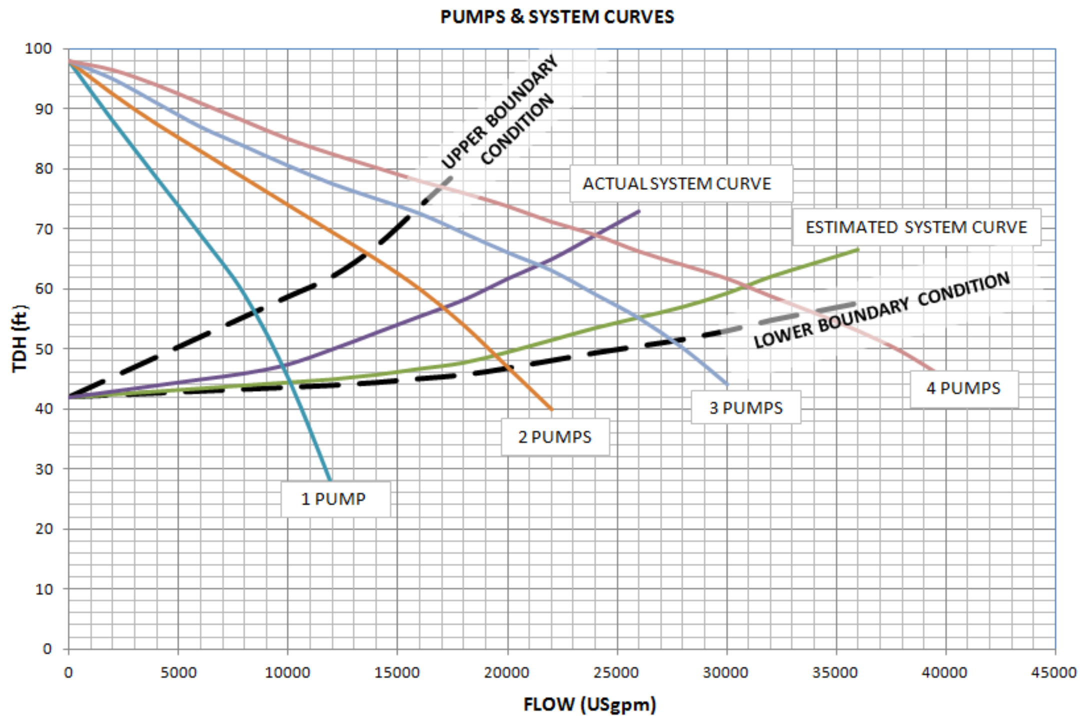

Figure 5: Define boundary conditions for existing system.

CONCLUSION

Conveyance capacity of aged pipelines might be significantly lower than the original capacity. As pipelines age, the conveyance capacity of the system may be reduced due to two primary reasons; ID reduction and increased roughness. Designers must evaluate the existing system and estimate the applicable ID reduction for future operation. The ID reduction can be estimated based on condition assessments of the existing pipeline. Field measurements are the most accurate method to estimate increased roughness. Designers must use accurate ID and roughness values to provide operational longevity, and avoid future flow capacity reduction.

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Combating Subsidence by Delivering Surface Water to Three Million Water Users—The Successes and On-Going Efforts

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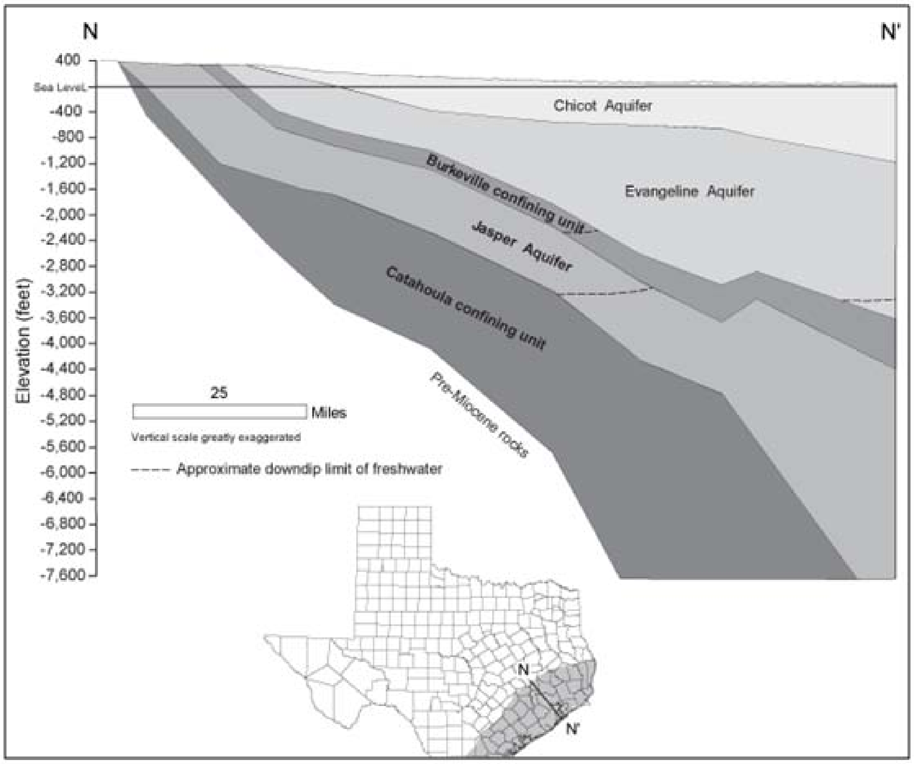
Abstract

This paper will benefit those interested in regional water supply planning to replace a substantial portion of existing groundwater supply. Excessive pumping of ground-water from an aquifer can greatly accelerate the consolidation of clay strata resulting in subsidence – the permanent loss of elevation at the surface or ground level. The authors outline the regional planning to substantially reduce subsidence in a heavily developed and rapidly growing region. Most attention is on the North Fort Bend Water Authority (Authority). The Authority's groundwater reduction plan (GRP) provides the benefit of regulatory compliance while reducing and equitably sharing the costs of compliance. To implement the GRP, the Authority has issued \$283 million in bonds to construct more than 50 miles (80 km) of 12- to 48-inch (30.5 to 122 cm) water lines including steel, bar-wrapped, PCCP, ductile iron, and PVC. Approximately 17 million gallons per day (mgd) (0.75 m^3/s) of surface water are delivered to 28 water plants throughout the Authority. The Authority successfully met the initial regulatory requirement to reduce ground-water use. Planning is ongoing to meet the 2025 deadline to further reduce ground-water use to no more than 40% of total water demand (i.e., a 60% reduction). The Authority is a participant in three regional water supply projects and will construct more than 30 miles (48.3 km) of water lines to complete its internal transmission system. The Authority's estimated cost for these four projects is more than \$700 million of the total estimated cost of more than \$3 billion.

BACKGROUND

The Gulf Coast Aquifer and Its Use

The Gulf Coast Aquifer (also, the Coastal Lowlands aquifer system) is a major aquifer paralleling the Gulf of Mexico coastline from Florida to Mexico. It consists of several aquifers, including the Jasper, Evangeline, and Chicot aquifers, which are composed of discontinuous sand, silt, clay, and gravel beds. Generally, the sand thickness of the Gulf Coast Aquifer ranges from 700 feet (213 m) in the south to 1,300 feet (396 m) in the north. Freshwater saturated thickness averages about 1,000 feet (305 m) (George, 2011). Figure 1 illustrates the important aquifers of the Gulf Coast Aquifer system.



Modified from Baker, 1979, 1986; Chowdhury and Mace, 2003; Kasmarek and Robinson, 2004

Figure 1 – Generalized Section of Strata Comprising the Gulf Coast Aquifer

Within the Houston-Galveston area, during the 40-year period beginning in 1935 and continuing through post-war industrialization until its peak in 1975, groundwater production increased dramatically from approximately 90 mgd (million gallons per day) (3.9 m^3/s) to a little more than 500 mgd (21.9 m^3/s) (Seifert, 2006).

The current population relying on the Gulf Coast aquifer just within the Houston region is more than 7 million (the population of the Houston consolidated metropolitan statistical area [CMSA]). Houston alone has a population of approximately 2.1 million.

Mechanics of Subsidence

High levels of groundwater production greatly reduce pressure within the aquifer and allow clay strata to consolidate. The consolidation of many feet of clay results in subsidence – the permanent loss of elevation at the surface or ground level. Figure 2 illustrates how subsidence occurs within the clay layers of the aquifer (Galloway, 1999).

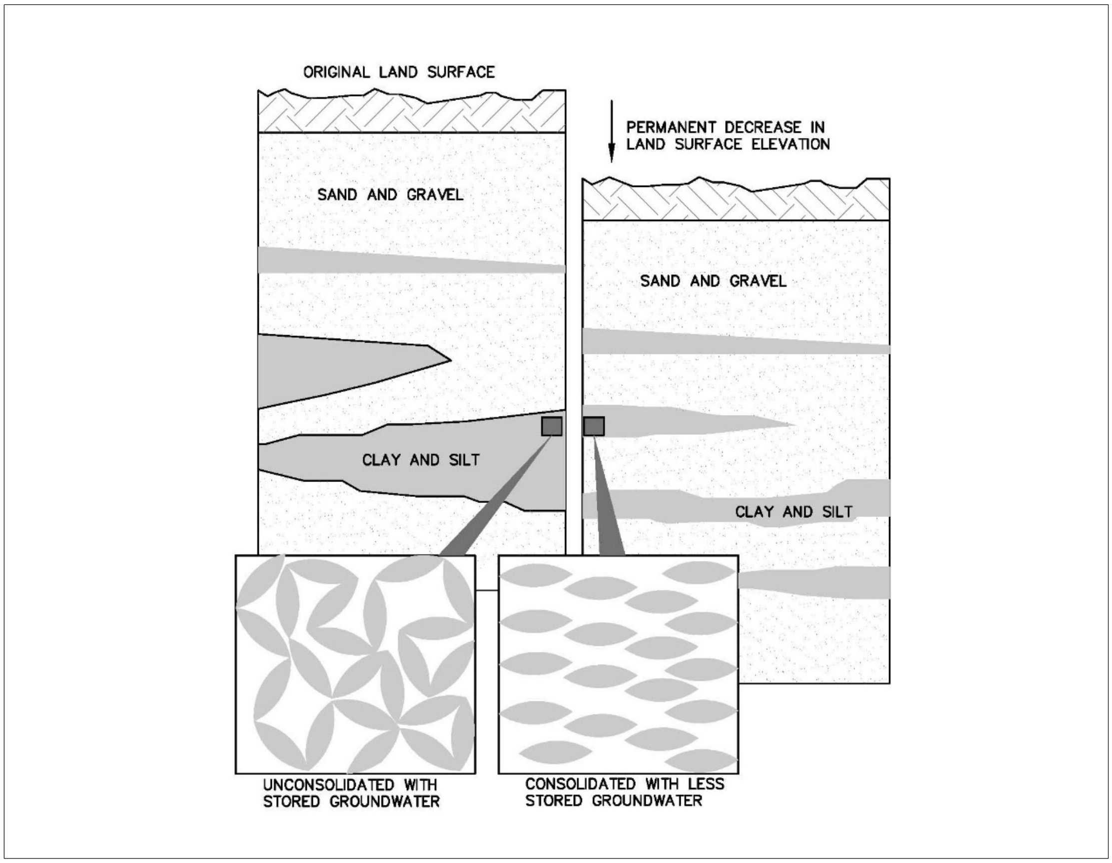


Figure 2 – Simplified Mechanics of Subsidence
(After Galloway, Jones, and Ingebritsen, 1999)

As far back as the 1920s, the Goose Creek oil field was the first place where subsidence of overlying terrain was attributed to the removal of oil from beneath the surface (Pratt, 1926) (Gabrysch, 1975). By the early 1940s, studies to identify problems due to groundwater extraction began. Original land-subsidence benchmarks, established just after the turn of the century, were re-leveled in the 1940s, and the results verified that subsidence was occurring.

In the 1950s and 1960s, community leaders linked the increased frequency and severity of flooding to subsidence. In the low-lying areas of Houston/Galveston – where tropical storms and hurricanes are a probability, not just a possibility – flooding was real and could be severe. In 1961, Hurricane Carla confirmed the worst fears about the impact of subsidence. Some water damage was not surprising given the severity of the storm, but the flooding that occurred was beyond what was expected from a hurricane of that size. As a result, local governments began to analyze the serious and very real impact that subsidence could have on the area's potential economic growth and quality of life, and, just as importantly, began to determine what exactly could be done about it.

With a number of studies linking groundwater withdrawal to subsidence – and ongoing measurements confirming those findings – groups of citizens began to work for a reduction in groundwater use in the late 1960s. In May of 1975, the Texas Legislature created the Harris-Galveston Subsidence District (HGSD), the first of its kind in the United States (Harris-Galveston Subsidence District, 2013).

The Legislature later created the Fort Bend Subsidence District (District) in 1989. Its purpose is to regulate the withdrawal of groundwater within the District to prevent subsidence that contributes to flooding, inundation or overflow of areas within the District, including rising waters resulting from storms or hurricanes (Tx Legislature, 1989).

Study and Documentation of Subsidence

The District has established a network of instruments to monitor subsidence using: extensometers, continuously operating reference stations (CORS), and periodically active monitor sites (PAMS) as shown in Figure 3. PAMS utilize a highly sensitive, trailer mounted (i.e. portable) GPS unit to occupy a site and record elevation every 30 seconds for 5 to 7 days every 2 months (Fort Bend Subsidence District, 2014). PAMS are the most cost effective and, therefore, most commonly used instrument in the network.

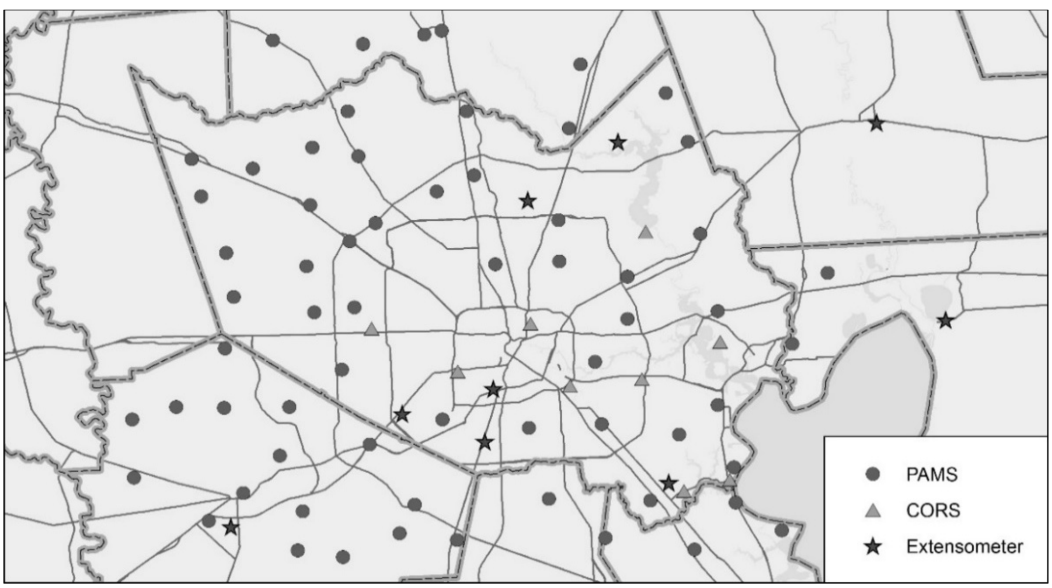


Figure 3 – Subsidence monitoring instruments operated by local subsidence and groundwater conservation districts in cooperation with the USGS.

Based on early conventional surveys and, later, the large volume of data collected using these instruments, the District has developed a graphic to depict subsidence in the Houston area using contour lines of feet of elevation loss since 1906 (see Figure 4) (Fort Bend Subsidence District, 2014).

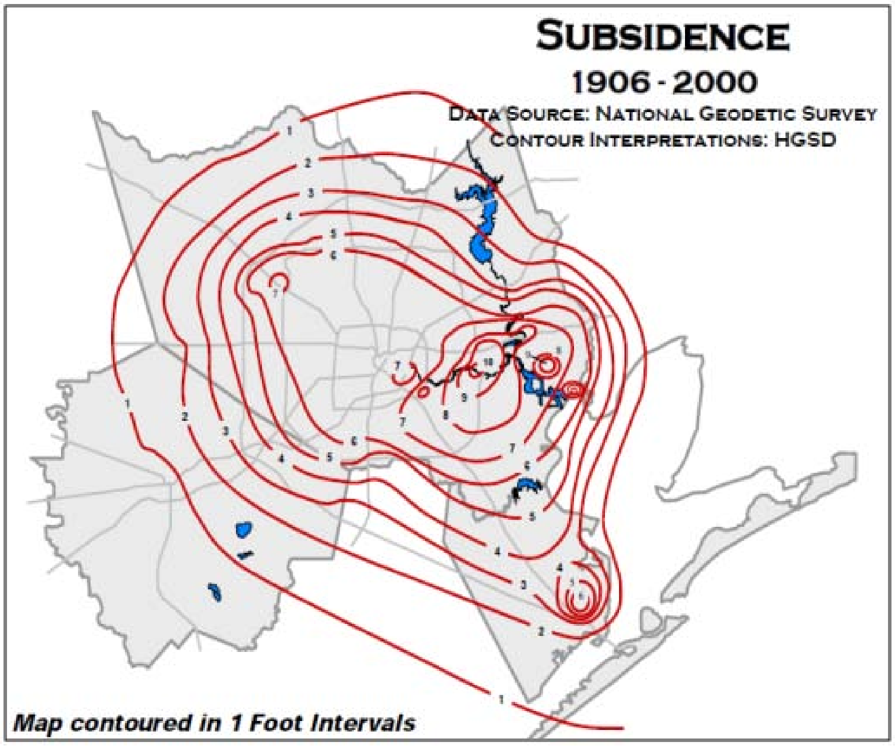
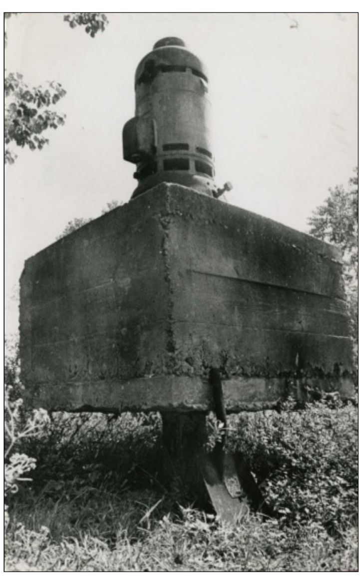


Figure 4 – Feet of elevation loss, 1906 - 2000

In addition to the regional scale of subsidence illustrated in Figure 4, there are smaller scale, tangible examples of subsidence. Figure 5 shows two examples of the effects of subsidence in the area of Baytown, Texas. The first example is a groundwater wellhead, where subsidence has left the concrete foundation suspended more than one foot above the ground. The second example shows a home in the abandoned Brownwood development after subsidence led to frequent inundation of the area.


Groundwater wellhead


House in abandoned Brownwood development

Figure 5 – Examples of subsidence near Baytown, Texas.

Groundwater Regulation

To address the effects of subsidence, the subsidence districts have developed rules requiring every owner of groundwater wells permitted to produce more than 10 million gallons per year (104 m^3/day) (and subject to other criteria) to develop and submit a groundwater reduction plan (GRP) for the district's approval. The primary element of the rules is that groundwater production is limited to some fraction of total water demand. Specifically, the Fort Bend Subsidence District (District) rules require that groundwater may constitute no more than 70% of total water demand in 2014 and no more than 40% of total demand by 2025. Stated differently, groundwater production must be reduced by 30% in 2014 and 60% by 2025. The reduction can be achieved through various means including substituting an alternative water source to satisfy demand. Other means include through use of treated wastewater effluent, conservation, and most recently through development of brackish waters. By far, the most important element of almost every GRP in the region is the use of surface water as an alternative to groundwater (Fort Bend Subsidence District, 2012).

To incentivize timely planning and adoption of alternative water sources, the District provides "credits" for excess or over conversion. Once earned, gallons of credit can be submitted in lieu of other alternative water, thus ensuring continued regulatory compliance in circumstances that might otherwise lead to a shortfall and violation of the required reduction in groundwater produced during a 12-month permit term.

SUCCESSES

Surface Water Supplies

The City of Houston (Houston) is the largest entity in the region subject to the groundwater reduction mandates of the subsidence districts. Because Houston's leaders have been acquiring and developing surface water supplies for more than 50 years, Houston holds the majority of surface water on which the GRPs rely.

In contrast, municipal utility districts (MUDs) are subdivisions of the State that are responsible for providing utility services. They also have the authority to levy property taxes to pay for utility construction. However, the more than 650 MUDs subject to the groundwater reduction requirements do not have access to adequate surface water to satisfy their needs and develop a GRP acceptable to the District, other than through Houston. Houston would have been challenged to work with so many MUDs, and the MUDs would have had great difficulty efficiently and cost effectively supplying the necessary water.

To address these challenges, State Representatives created water authorities. The North Harris County Regional Water Authority (NHCRWA, population 700,000) was the first to be created in 1999. The other three authorities include Central Harris County Regional Water Authority (CHCRWA, 2001, population 50,000), West Harris County Regional Water Authority (WHCRWA, 2003, population 500,000), and North Fort Bend Water Authority (Authority, 2005, population 200,000). The authorities have the powers necessary to act on behalf of the MUDs within their territory. The most essential of those powers include securing alternative water supplies and charging rates and fees necessary to pay for infrastructure to deliver the water. None of the authorities have the power to levy taxes.

The North Fort Bend Water Authority

The Authority serves as an example of how the authorities have met the regulatory needs of the numerous MUDs within their territories by acquiring surface water from Houston, planning the infrastructure to deliver it, and obtaining the necessary funding to finance that infrastructure. Since its creation in 2005, the Authority has succeeded in its implementation of its plans summarized in the following paragraphs.

Soon after creation, the Authority began studies of future population and water demand (Brown & Gay Engineers, Inc., 2013), a "source study" to identify potential sources of alternative water available to the authority, and an "alternative analysis" to evaluate alternative delivery systems and operational strategies to minimize cost. The results of all three studies fulfilled required elements of the GRP submitted to the Fort Bend Subsidence District.

The Source Study concluded that Houston was the only entity with sufficient water to supply the future needs of the Authority (Brown & Gay Engineers, Inc., 2006). The Alternative Analysis required development of hydraulic models and a "cost model" to evaluate the delivery systems and operational strategies and select the alternative recommended to the Board of the Authority (Brown & Gay Engineers, Inc., 2007). Both models continue to be updated during growth of the system and, as projects are funded, to evaluate long term effects on water rates.

The Authority's GRP was submitted in 2008 ahead of the District's deadline. The GRP provided regulatory compliance for MUDs (and other groundwater well owners subject to reduction requirements) by identifying the estimated future population and water demand, the source(s) of future water including a contract for that water, and the proposed infrastructure and its estimated capital cost. The Authority's GRP anticipates developing the lowest cost system by minimizing the miles of water line installed. Minimizing miles of water line is achieved by over-converting some MUDs (i.e., deliver more than the regulatory requirement) for the benefit of regulatory compliance for all MUDs and for which all participants in the GRP share the cost.

Implementation

Once the GRP was in place, the Authority began the steps necessary to implement the plan. Major steps in implementation included obtaining the necessary funding, acquiring the right of way, and developing design guidelines for consultants to follow as well as standard construction documents (contract documents, technical specifications, and standard detail drawings).

The Authority issued bonds in 2009, 2010, and 2011 totaling $283 million. Funds have been used to maintain the Authority's operations, with the majority of funds designated for the planning, design, construction, and testing of 50 miles (80.5 km) of water line. Water lines constructed to date vary from 48-inch (122 cm) transmission lines down to 12-inch (30.5 cm) water lines connecting the MUDs' water plants. Construction of water lines began in 2009 to achieve 30% reduction in groundwater by 2014, and construction was completed in 2014. The Authority began to deliver surface water in March 2011 by leasing a 10 mgd (0.4 m^3/s) water plant from Houston. The Authority's permanent pump station began service in May 2014. From March 2011 through September 2014, all surface water delivered accrued approximately 10.9 billion gallons (41.3 million m^3) of credit for future use if needed.

Water Rates

The Authority charges a fee on groundwater pumped, the pumpage fee, and a separate rate for surface water delivered to MUDs. The pumpage fee began in October 2005, at $0.19 per thousand gallons pumped ($0.19/1000 gal, $0.05/ m^3) and has increased about $0.30/1000 gallons ($0.08/m^3) annually as the Authority's debt service, operation and maintenance (O&M) costs have increased.

In addition to the pumpage fee, the Authority charges a rate on surface water set at the pumpage fee plus $0.35/1000 gallons ($0.09/m^3). The difference between pumpage fee and surface water rate offsets the costs associated with producing groundwater so that there is no incentive or disincentive to receive surface water.

The Authority's current groundwater pumpage fee effective January 1, 2015, is $2.45/1000 gallons ($0.65/m3) and the surface water rate is $2.80/1000 gallons ($0.74/m3).

ON-GOING EFFORTS

2025 Regulatory Deadline

The Authority is looking ahead to the next Subsidence District milestone in 2025, which requires an additional 30% reduction (60% total) in total groundwater use. The infrastructure projects required to treat and convey large quantities of surface water are some of the largest projects in the country.

Luce Bayou Interbasin Transfer Project

The Luce Bayou Interbasin Transfer Project (LBITP) consists of pump station, pipe line, and canal to convey raw water from the Trinity River basin to the San Jacinto River basin. The ultimate capacity of the project is approximately 500 mgd (21.9 m^3/s). The pipe line portion of the project consists of approximately 3 miles (4.8 km) of parallel 108-inch (274 cm) diameter water line. After discharging to a sediment basin, flow continues through approximately 23 miles (37 km) of canal to Lake Houston, an integral part of Houston's raw water supply. Total cost of the project is estimated to be $434 million of which the Authority's share is $72 million. Figure 6 shows the main components of the LBITP project.

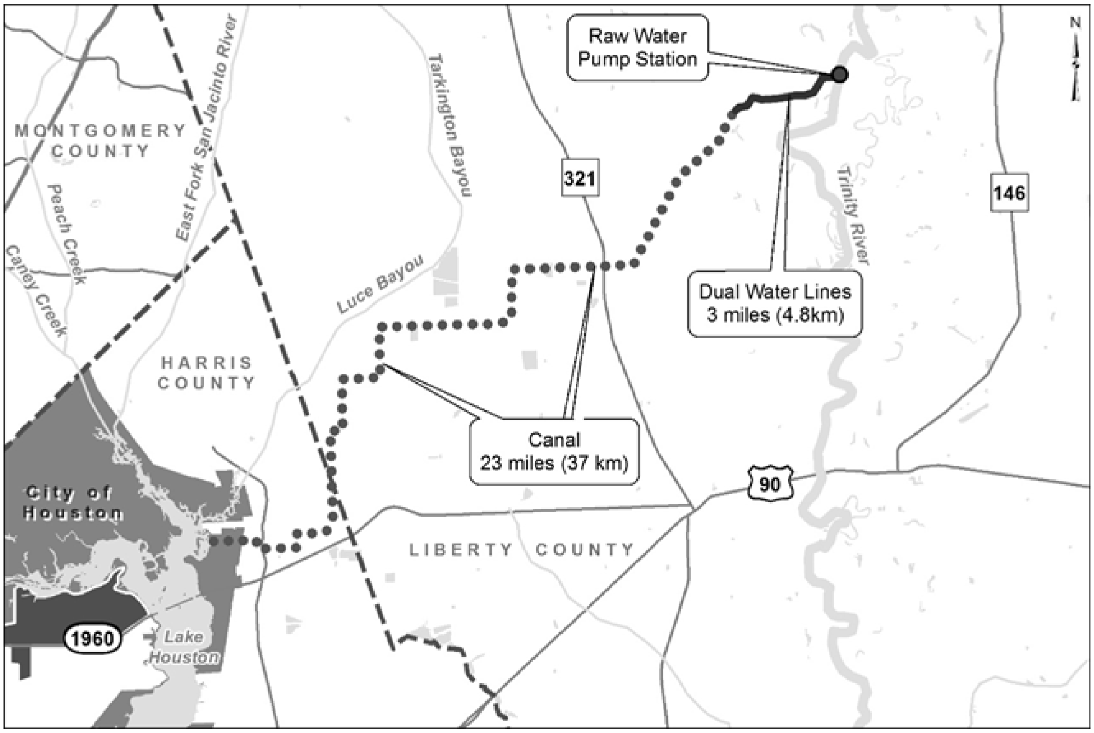


Figure 6 – Luce Bayou Interbasin Transfer Project

North East Water Purification Plant Expansion Project

Houston's exiting Northeast Water Purification Plant (NEWPP) does not have adequate capacity to supply the future needs of Houston and the water authorities. Houston and the authorities entered into an agreement in February 2015 to construct a 320 mgd (14 m^3/s) Expansion Project on the existing plant site adjacent to existing treatment facilities. The Project consists of new raw water intake, electrical substation, and treatment facilities for a first phase of 80 mgd (3.5 m^3/s) to be complete in 2021 and second phase of 240 mgd (10.5 m^3/s) to be complete in 2024. Total cost of the project is estimated to be $1.28 billion of which the Authority's share is $300 million. Figure 7 shows the location of the NEWPP Expansion project.

Second Source Water Transmission Line Project

The Second Source transmission line is a joint project between the WHCRWA and the Authority. The transmission line is a 40-mile (64 km), 160 mgd (7 m^3/s) capacity, 96-inch (244 m) water line beginning at the NEWPP Expansion described, above, and includes two booster pump stations. Flow will be measured at a meter station where the transmission line crosses from the WHCRWA to the Authority. Total cost of the project is estimated to be $700 million of which the Authority's share is $300 million. Figure 7 shows the alignment of the Second Source project.

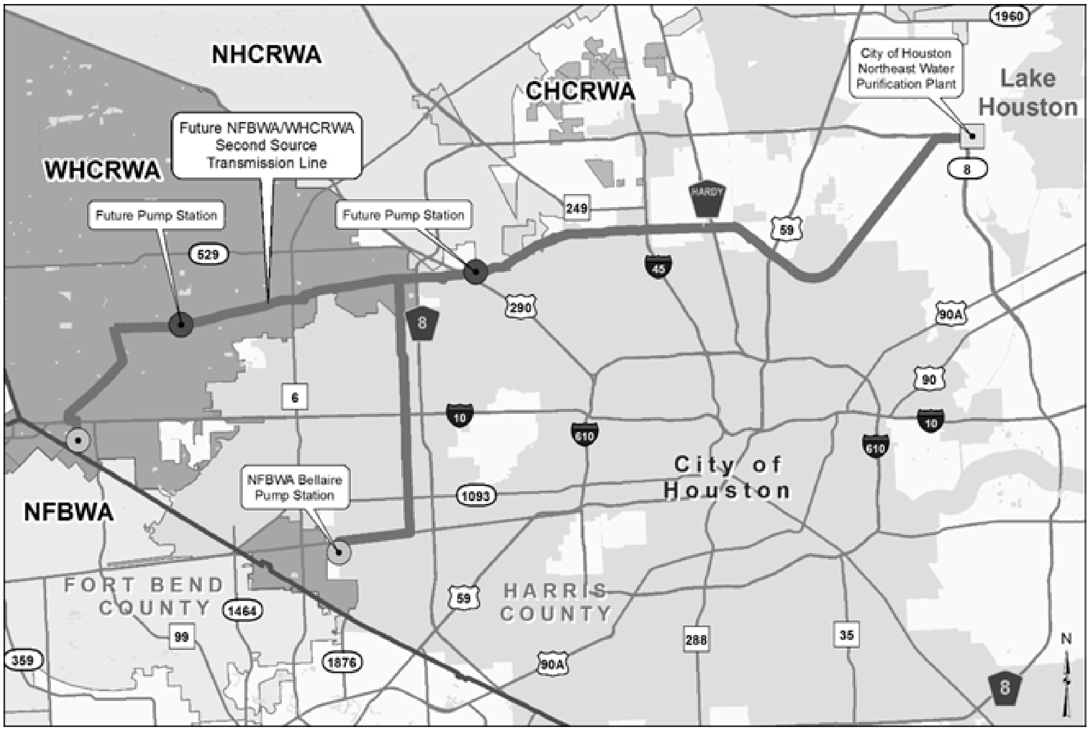


Figure 7 – NEWPP Expansion Project and Second Source Transmission Line

Authority Internal Water Transmission Projects

Treated surface water is finally transmitted to MUD water plants in the Authority that are part of the 2025 phase of conversion to surface water. The planned water lines connect to existing transmission lines constructed as part of the 2014 conversion. The completed system of transmission lines creates a "looped" system to provide redundancy and ensure reliability of the system.

The 2025 phase transmission projects consist of 30 miles (32 km) of 60-inch (152 cm) to 24-inch (64 cm) large diameter transmission mains plus smaller water lines connecting to water plants owned and operated by the MUDs within the Authority. The MUDs still to be supplied surface water in the future include the single largest water user in the Authority as well as some of the largest and fastest growing developments in the north and far west areas of the Authority.

The total cost of these Authority projects is estimated to be $200 million. Figure 8 shows the planned water lines comprising the Authority's 2025 internal water transmission lines.

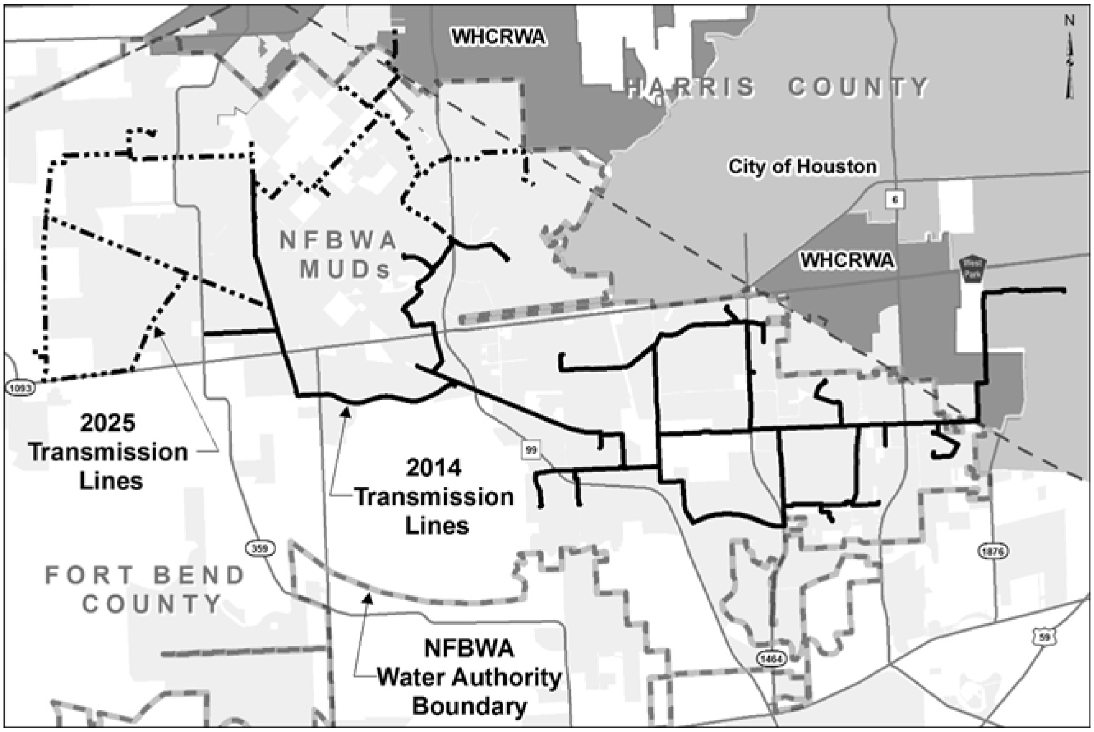


Figure 8 – Authority Existing Transmission Lines and 2025 Internal Water Transmission Lines Project

SUMMARY AND CONCLUSIONS

The first signs of subsidence due to human activity were evident almost a century ago. Local and State leaders have been united in their response that groundwater production must be monitored and reduced to a sustainable level to avoid the widespread, adverse impacts of subsidence. Subsidence districts were created to perform the scientific investigation needed and to set regulatory requirements.

The North Fort Bend Water Authority accomplished the necessary engineering and construction to achieve the required 30% reduction in groundwater use by 2014. The 30% reduction in demand on groundwater resources satisfies the regulatory requirement for more than 200,000 people in northern Fort Bend County. The Authority must complete significant infrastructure at an estimated cost of $700 million to achieve the additional 30% reduction (60% total) by 2025.

The combined population of Houston and the regional water authorities is more than 3 million people that will be supplied by large regional projects, which are estimated to cost $3 billion.

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Flow-Based Modeling for Enhancing Seismic Resilience of Water Supply Networks

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Abstract

Modern urban societies depend greatly on water supply systems for economic prosperity, public health, security, and quality of life. Uninterrupted water supply is even more important during times of natural calamities such as earthquakes. Seismic hazards during past earthquakes have caused significant damage to buried pipelines rendering the water systems dysfunctional. This paper presents a flow-based metric for assessing and enhancing water supply resilience against the seismic hazard of liquefaction-induced settlement. The proposed resilience metric is demonstrated considering a water supply network in a coastal region in South Carolina. Seismic settlement is evaluated using an available liquefaction potential map. Evolutionary optimization techniques are employed to identify the optimal set of pipeline replacement strategies to enhance seismic resilience of the selected water supply network. The computational tools employed include the *igraph* package in R software, the *EPANET* and its toolkit, and the MATLAB programming environment. The presented framework along with the results will be useful to utility departments that manage pipeline systems in earthquake-prone regions.

1. INTRODUCTION

Water supply networks (WSNs) are one of the critical infrastructures whose disruption following an earthquake will cause significant inconvenience to people, as evidenced from past earthquakes such as Charleston-1886, San Francisco-1906, Northridge-1994, Kobe-1995, Haiti-2010, and Christchurch-2011. Continuous functioning of WSNs is also crucial for firefighting and operation of medical facilities following an earthquake. Therefore, it is important for water utility operators in earthquake prone regions to evaluate their WSN's resilience against seismic hazards and develop appropriate rehabilitation plans and emergency preparedness strategies.

There have been a number of studies on seismic resilience evaluation of WSNs. Bruneau et al. (2003) characterized seismic resilience of communities as "the ability of the system to reduce the chances of a shock, to absorb a shock if it occurs, and to recover quickly after a shock." Their resilience framework encompasses four dimensions, namely robustness, redundancy, rapidity and resourcefullness. Several studies employed frameworks that are similar to that of Bruneau et al. for evaluating the resilience of infrastructure systems such as WSNs against natural or manmade hazards. For example, Chang and Shinozuka (2004) combined the framework presented by Bruneau et al. (2003) with a loss estimation model to develop quantitative measure of resilience and demonstrated it in mitigating seismic consequences of a WSN in Memphis, TN. Other notable recent studies on seismic hazards and WSN performance evaluation include: Romero et al. (2010), which presented the simulated response of Los Angeles's WSN against a 7.8 moment magnitude earthquake; Fragiadakis et al. (2013), and Fragiadakis and Christodoulou (2014), both of which evaluated the seismic reliability of WSNs; and Laucelli and Giustolisi (2014), which adopted a risk assessment approach for analyzing the vulnerability of a WSN.

This paper presents an easy-to-use metric to estimate resilience of WSNs against any given hazard, and demonstrates its use by characterizing the resilience of a WSN that serves part of the City of Charleston in South Carolina to liquefaction-induced settlement. The proposed resilience metric considers pipeline failure probabilities in addition to topological and hydraulic behavior of WSNs. Because the metric considers hydraulic flows, it is called the flow-based resilience metric in this paper. A simple optimization problem is framed and solved to identify the best set of rehabilitation strategies that will enhance resilience of the chosen WSN.

2. FLOW-BASED RESILIENCE METRIC

Resilience is defined in this paper as the ability of the system to continue to satisfy demands under a given perturbation. A resilient WSN is characterized in this paper as one that encompasses two specific characteristics, namely robustness and redundancy. Robustness is the ability of the WSN to withstand stresses imposed on it with little or no loss in capacity, while redundancy refers to the buffer capacity available to meet the system needs in case of component failure(s) and increased demand(s). Buffer capacity can be attained in the form of additional energy (i.e., pressure) available for dissipation in case of failures, and topological connectedness of the network. If a pipeline is able to resist a specific type of abnormal loading, it is said to be robust against that type of loading. If a given node in the WSN is connected to multiple pipelines and is served with more pressure than required, it is redundantly connected to the source and also able to compensate for pressure loss from any unforeseen events.

Based on features of resilience described above, the following metric is proposed in this study to quantify WSN resilience:

$$R = \frac{\sum_{t=1}^{td}\sum_{i=1}^{N_n}\left(\left(\sum_{j=1}^{N_i}(1-P_{fj})\right)q_{i,t}^{*}(h_{i,t}-h_{i,t}^{*})\right)}{4\times\sum_{t=1}^{td}\sum_{i=1}^{N_n}q_{i,t}^{*}h_{i,t}^{*}} \tag{1}$$

where R = the flow-based resilience metric; td = number of time steps in the demand pattern of WSN; N_n = number of nodes in WSN; P_{fj} = failure probability of link j connected to node i; N_i = number of links connected to node i; $q_{i,t}^{*}$ = demand of node i in time step t; $h_{i,t}$ = actual total head at node i in time step t; and $h_{i,t}^{*}$ = minimum required total head at node i in time step t. Equation 1 captures the connectedness of WSN nodes and failure probabilities of respective connected pipelines in the form of $\sum(1-P_{fj})$. It accounts for hydraulic buffer capacity available at each node in terms of surplus nodal head, $(h_{i,t}$ - $h^*_{i,t})$. It also gives priority to nodes with greater demand by multiplying the numerator with $q^*_{i,t}$. The term $\sum q^*_{i,t} h^*_{i,t}$ is added to the denominator in Equation 1 to ensure that the range of R will be between 0 and 1 in a majority of cases.

The metric defined by Equation 1 is employed in this paper to characterize resilience of the Charleston WSN to liquefaction-induced settlement.

3. FAILURE PROBABILITY DUE TO LIQUEFACTION SETTLEMENT

The methodology to calculate pipeline failure probability (P_f) against liquefaction-induced ground settlement is presented in this section. Liquefaction ground failure, in the form of ground settlement and lateral spreading, has been a predominant source of pipeline failures in past earthquakes (Cubrinovski et al., 2014).

One approach for estimating liquefaction-induced ground settlement is described in American Lifelines Alliance guidelines (ALA 2001, 2005). Based on this approach, pipeline failure probability is estimated by (Su et al., 1987; Piratla and Ariaratnam, 2011; Fragiadakis et al., 2013):

$$P_f = 1 - e^{-RR*L} \tag{2}$$

where L is pipeline length in 1,000 feet (1 foot = 0.3048 m); and RR is estimated repair rate (number of repairs/1,000 feet of pipeline). ALA (2001) provides the repair rate per 1000 ft of pipe length due to ground settlement as:

$$RR = K \times 1.06 \times S^{0.319} \tag{3}$$

where S is settlement in inches; and K is a correction factor to account for differential behavior of various pipe materials to settlement hazard. Typical values of K are presented in Table 1.

Table 1. Correction factor K for adjusting RR (Adapted from ALA, 2001)

Pipe Material	K
Cast Iron and Asbestos Cement	1.0
Welded Steel	0.7
PVC	0.8
Ductile Iron	0.5

According to ALA (2005), settlement can be roughly estimated based on liquefaction susceptibility or potential as shown in Table 2. Liquefaction potential represents the likelihood of liquefaction occurring in a particular area for a given earthquake loading.

Table 2. Approximate settlement based on liquefaction susceptibility or potential (Adapted from ALA, 2005)

Liquefaction Susceptibility or Potential	Settlement, S (inches)
Very High	8.1
High	3.9
Low to Moderate	1.9

4. DEMONSTRATION OF RESILIENCE FRAMEWORK

A section of the WSN that serves the Charleston peninsula region, hereafter referred as the Charleston City WSN or just the WSN, is chosen as a test bed to demonstrate the use of the proposed flow-based resilience metric. The skeleton structure of the Charleston City WSN, which can be seen in Figure 1, has been obtained in the GIS format from the corresponding water utility operator (Piratla et al., 2014).

The Charleston City WSN has a total of 1,483 demand nodes and 1,896 links with an approximate length of 110 miles (1 mile ≈ 1.609 km), and consists of ductile iron (69%) and gray cast iron (22%) for pipeline materials for the most part measured by length. The WSN pipeline sizes range from 6" to 24" in diameter, with 6" constituting about 57% measured by pipeline length. Using available WSN data such as, pipe lengths, diameters, locations, and respective connectivity, a hydraulic model for the WSN was designed in this study by appropriately adding one reservoir, few pumps and tanks using the EPANET2 hydraulic solver.

4.1. Ground Settlement

Charleston is prone to seismic hazards such as liquefaction-induced settlement with potentially devastating consequences as evidenced from the 1886 Charleston earthquake. This earthquake was the most damaging earthquake to have occurred in the southeast United States, causing 124 deaths and approximately $540 million (2014 dollars) in damage (Côté, 2006).

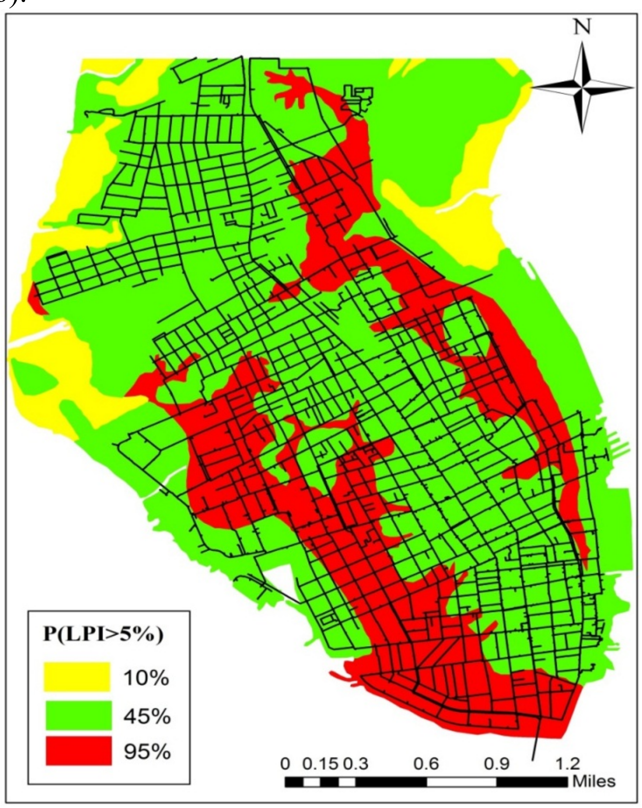

Figure 1. The Charleston City WSN plotted on the liquefaction potential map by Hayati and Andrus (2008)

The liquefaction potential map created by Hayati and Andrus (2008) for the WSN region is also presented in Figure 1. This map is adopted for estimating the liquefaction-induced settlement. The map is based on the liquefaction potential index (LPI) proposed by Iwasaki et al. (1982). As can be seen from Figure 1, the study region is divided into three zones: (1) 95% average probability of exceeding LPI of 5 ($P_{LPI>5}$); (2) 45% of $P_{LPI>5}$; and (3) <10% of $P_{LPI>5}$. A value of 5 for LPI is considered a threshold for the generation of sand boils caused by liquefaction (Hayati and Andrus, 2008). The liquefaction potential map was developed assuming a moment magnitude of 7 and a peak ground acceleration of 0.3g, which represent the approximate loading conditions during the 1886 Charleston earthquake.

4.2. Failure Probability

Failure probability due to liquefaction settlement can be estimated from Equation 2 by: (1) assuming $P_{LPI>5}$ values of 95%, 45% and <10% correspond to liquefaction potentials of very high, high and low to moderate; and (2) estimating K and S using Tables 1 and 2, respectively.

Figure 2 illustrates the WSN layout depicting pipelines with corresponding computed failure probabilities. As can be seen from Figure 2, failure probabilities are grouped into four categories: 0-10%, 10-45%, 45-90%, and 90-100%. As can be seen in Figure 2, the majority of pipelines (total length adding up to 78 miles or 125.5 km) are located in 10-45% failure probability area while 30 miles (or 48.3 km) of pipelines are located in the areas with failure probability of more than 45%.

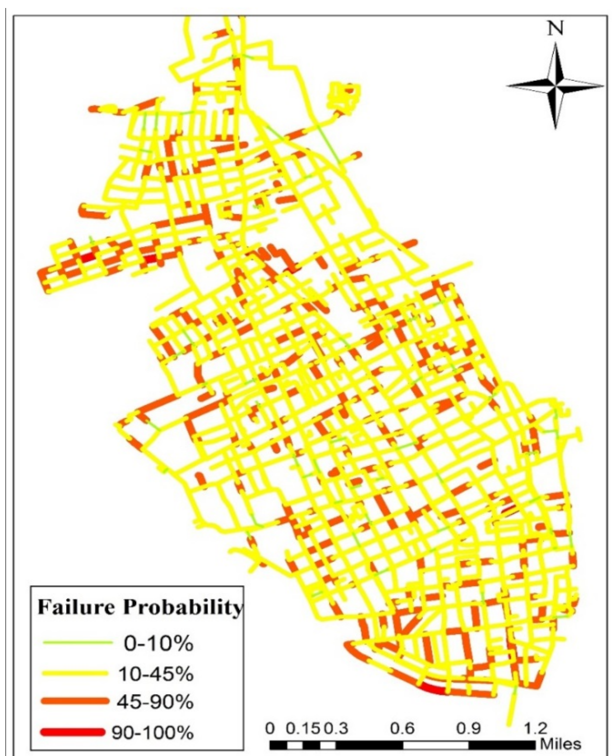

Figure 2. Pipeline failure probability (P_f) of the Charleston city WSN

4.3. Resilience to Liquefaction Settlement

Based on nodal demands, topology, and calculated pipeline failure probabilities, resilience of the WSN to liquefaction-induced settlement is estimated using the metric presented in Equation 1. The EPANET toolkit in MATLAB programming environment is employed to calculate nodal heads ($h_{i,t}$) in different time steps. The *igraph* library package of R software (Csardi and Nepusz, 2006) is used to calculate weighted (i.e., $\sum(1 - P_{fj})$) topological node degree. Shown in Figure 3 is the assumed 24-hour demand pattern based on the trend presented in Ciaponi et al. (2011). The value of R against settlement for the WSN is estimated to be 0.35.

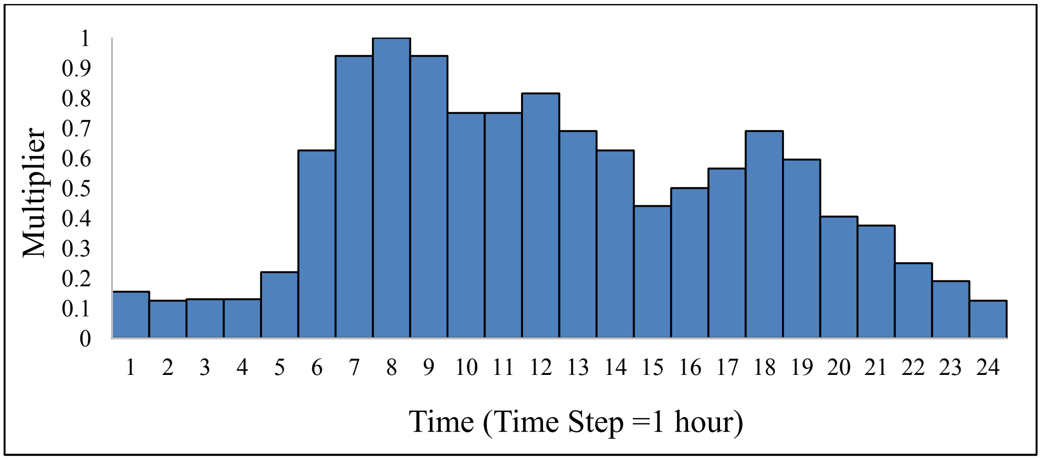

Figure 3. Demand pattern used for hydraulic simulation of the WSN

The calculated value of R for the WSN is best understood by comparing it to calculated R values for different scenarios. Four such scenarios are considered: (1) all pipeline failure probabilities are zero (S_1), which represents a perfectly *robust* WSN where pipelines are capable of resisting the liquefaction settlement hazard without failing; (2) all nodes are supplied with 10% greater head (S_2), which represents a more *redundant* WSN where a component failure is better handled due to greater buffer energy available at each node; (3) all pipeline failure probabilities are zero and all nodes are supplied with 10% greater head (S_3) which represents a perfectly *robust* and more *redundant* WSN and (4) 10% of the WSN pipeliens are randomly removed (S_4), which represents a less *redundant* WSN where fewer paths exist between source and demand nodes. One thousand simulations are conducted to estimate the average resilience value for S_4 scenario. The R values for these four scenarios (S_1, S_2, S_3, and S_4) along with the base scenario (S_b) are presented in Figure 4.

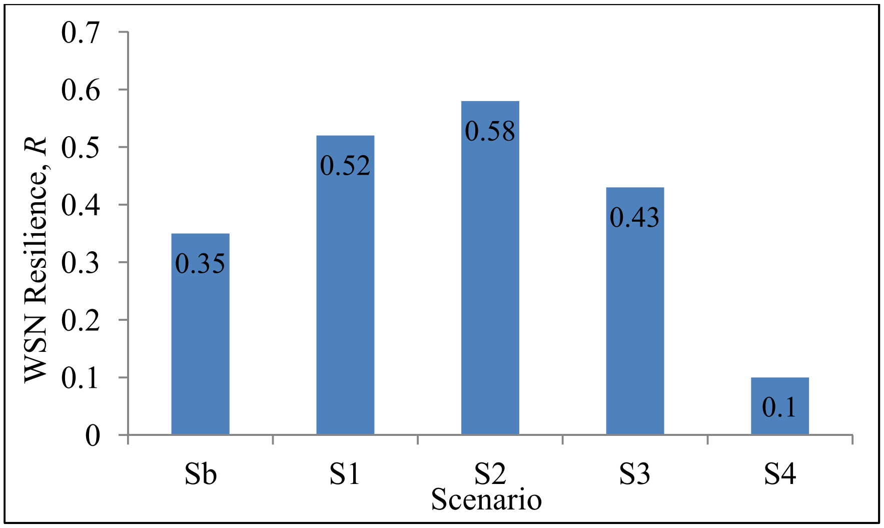

Figure 4. WSN Resilience in Different Scenarios

It can be observed in Figure 4 that the WSN resilience in S_b is about 67% of that in S_1, a perfectly robust WSN, and 60% of that in S_2. Similarly, the WSN resilience in S_b is about 81% of that in S_3, and 350% of that in S_4. Although some of these percentages will vary with changes in considered nodal head increments (in S_2 and S_3) and percentage of pipelines eliminated (in S_4), they nevertheless reflect the WSN resilience.

It should be noted that achieving a value of 1 for the WSN resilience as defined in this study is highly unlikely, because it requires on average that all pipeline failure probabilities are zero, all nodes are connected to four or more pipelines, and the buffer head at each is equal to the minimum required head.

4.4. Resilience Enhancement

Several rehabilitation actions can be taken to enhance WSN resilience using the proposed resilience metric as the basis. They include but not limited to replacing vulnerable pipelines with earthquake-resistant pipelines, adding new pipelines where possible to build topological redundancy, adding energy redundancy to the system, and cleaning and lining of pipelines. By considering the pipeline replacement option alone, an optimization problem is defined and solved to identify the optimal set of pipelines that should be replaced for greatest resilience enhancement, while being subject to budget constraints.

Flexible pipe materials such as ductile iron and High Density Poly Ethylene (HDPE) have performed well in the past earthquakes (Lund, 1996; Cubrinovski et al., 2014; and Piratla et al., 2014). Consequently, all non-ductile iron pipelines were considered as potential candidates (i.e., decision variables in the optimization problem) for replacement with flexible pipe materials. In order to meet the growing urban demands of the WSN, diameters of replaced pipelines are increased by four inches. Additionally, the roughness coefficient (C) of replaced pipelines is changed to 140 in the hydraulic simulation. The EPANET2 (Rossman, 2000) software which is a computer program that analyzes hydraulic behavior of WSNs, was used for changing system parameters (e.g., diameter value, roughness value, etc.) and calculating hydraulic performance outputs such as pipeline flows and nodal heads ($h_{i,t}$) in each time step.

The value of R is considered as the objective in the optimization problem, while total cost of pipeline replacement is the constraint. Replacement cost of pipelines was obtained from literature assuming that pipe bursting technique will be used. A replacement cost of \$8/inch/ft (1 inch = 25.4 mm) is assumed after appropriately adjusting for inflation (Boyce and Bried, 1998). Binary genetic algorithm in the MATLAB programming environment is used to solve this optimization problem. EPANET toolkit is used to integrate the optimization algorithm in MATLAB with the EPANET2 hydraulic solver.

Resilience can be improved in two ways with the replacement option: (a) by reducing P_f, and (b) by increasing $h_{i,t}$. Failure probability is reduced when non-ductile iron pipe materials are replaced with flexible materials because of reduction in K (Table 1), while actual node head is increased when larger diameter pipelines are used at appropriate locations. The optimization problem is solved for different budget constraints, and Figure 5 illustrates the resulting resilience improvement. Several smaller diameter pipelines in regions of $P_{LPI>5}$=45% were chosen for replacement. It has been observed that resilience has increased in a steep manner from 0.35 to 0.45 before it became asymptotic to the maximum possible value for the WSN. Resilience increased by about 27% with the first \$5 million budget, and overall about 37% with \$13 million investment. These results provide some guidance to the utility operator in capital improvement planning.

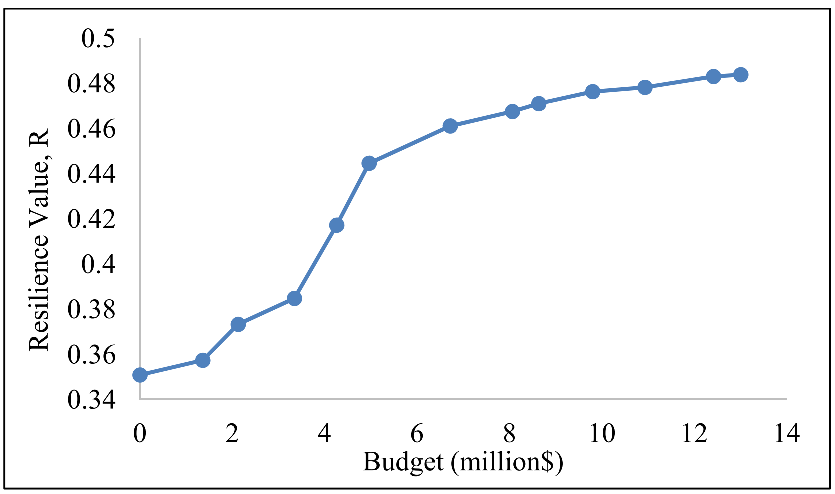

Figure 5. Tradeoff between resilience and cost in the resilience enhancement process

5. CONCLUSIONS

Past earthquakes have caused severe damage to buried water pipelines resulting in disruption of WSNs and serious trouble in daily life of people in disaster areas. It is therefore imperative to assess the risk associated with seismic hazards and take measures to enhance the resilience of WSNs. This paper proposed a new metric to evaluate resilience of WSNs and demonstrated its use to quantify the resilience against liquefaction induced settlement of a WSN serving Charleston, South Carolina. The metric was also used for identifying the optimal set of pipeline replacement options for enhancing the WSN resilience against settlement.

The settlement resilience value of the studied WSN was found to be 0.35, which is about 67% of a perfectly robust WSN. Several pipeline replacement options were evaluated for enhancing resilience using a binary genetic algorithm optimization tool in the MATLAB programming environment. The optimization results revealed that resilience of the WSN can be improved by 27% with a budget of about $5 million, and the improvement could be upto 37% with an overall budget of about $13 million.

Limitations of this study include: (a) the lack of an accurate hydraulic model for the study area and the subsequent use of a theoretical flow model; (b) the lack of consideration of rehabilitation types such as pipe lining and cleaning in the optimization problem that was framed and solved; and (c) the assumptions made in developing the liquefaction potential map in Figure 1, including the assumption that older deposits are more resistant to liquefaction than younger deposits and the open cut pipeline installation process did not result in lower liquefaction resistance. Further research is needed to characterize the influence of earthquake magnitudes and ground acclerations on the resilience of Charleston WSN. In the future, the resilience analysis presented in this paper should be integrated with reliability-based rehabilitation planning after taking into account demand forecasts in addition to pipe age, break frequencies, and subsequent criticalities.

6. ACKNOWLEDGMENT

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Benefits and Lessons Learned from Implementing Real-Time Water Modeling for Jacksonville Electric Authority and Western Virginia Water Authority

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Abstract

There are countless possible benefits of real-time modeling, including model calibration improvement, advanced alerting of system abnormalities, forensic analysis, and real-time evaluation and predictive optimization of water quantity, quality and energy usage. All the pieces are available to connect real-time water system information to hydraulic models, making these benefits a reality. The pieces include advanced computing hardware and software, accurate GIS and hydraulic models, increasing amounts of real-time data about water system performance, and database and communication protocols ready for integration. The process of integrating and implementing a real-time modeling system can seem daunting, especially without the right tools, and one must consider if the value and potential improvements are worth it. The tools are increasingly available, and water systems are beginning to see the potential values of real-time implementation. This paper will discuss the benefits of real-time modeling, implementation challenges and successes, as well as two successful implementations at a medium and large-sized water systems. In particular, these water systems implemented the Innovyze IWLive real-time modeling software platform. CH2M HILL provided implementation assistance and coordination between the water system and the software provider, Innovyze. Some of the challenges included incoming data quality, database integrity and robustness, hydraulic model readiness, and each of these will be discussed, along with the solution to achieve success.

BACKGROUND

Real-time hydraulic modeling is the process of linking a hydraulic model to available operational supervisory control and data acquisition (SCADA) data streams so that the hydraulic model is continually running simulations on a periodic basis. The periodic basis can vary from every minute, every hour, every day, etc. depending on the needs and applications of the utility, but the general concept is being able to establish model runs and thus access model results based on the most recent field data available. By having implemented real-time modeling, a utility is able to make more informed decisions since the modeling tool is more accurate and relevant being based on the latest boundary conditions. Plus the confidence in the hydraulic model results is greatly improved since the model is continually being validated (comparing the most recent field conditions to the model results). The applications and benefits are many from advanced system abnormality alerts, predictive optimization (water quality, quantity, and energy use), emergency response planning, forensic analysis, operator training, and much more. In the upcoming years, real-time modeling practice will continually grow in our industry* and will dramatically impact how hydraulic models are developed and applied. It is important now to establish the framework for implementation that may provide a standard procedure for other utilities that are planning to migrate to a real-time modeling platform.

METHODS

The steps for implementing a real-time hydraulic model can vary from utility to utility based on the available data, resources, and planned application(s) of the model. We will discuss the common implementation procedures based on lessons learned from two successful case studies of implementing real-time modeling for Jacksonville Electric Authority (JEA) and Western Virginia Water Authority (WVWA). The common steps that were found with implementing both of these real-time hydraulic models were the following: developing an operational hydraulic model, establishing SCADA data access, configuring SCADA tag mapping to the hydraulic model facility elements, identifying demands and demand patterns to be used during simulations, and validation/calibration of the real-time model. Ancillary configurations may include developing warning templates, dashboards, and mapping themes as well as setting up weather forecast and demand prediction data feeds.

Step one for establishing a real-time hydraulic model is developing an operational hydraulic model that reflects the changing conditions of a system in terms of operational controls and demands. The sub-steps involved are establishing that the model is representing the majority of the system to accurately provide results that reflect the real-world. As with all hydraulic models, the level of detail is based on the desired application. The utility that is primarily concerned with water quality will want a more detailed and accurate model in terms of modeling majority of the pipes, valves, and nodes versus the utility that may only be interested in optimization of their energy use will not require such a detailed model representation. An important aspect to consider for any application is establishing a procedure to update the model when assets are changed (upgraded, removed, replaced, added, etc.) whether that is integration with GIS or manual, periodic updates to the hydraulic model. Proper pressure zone delineation should also be a major condition in prepping an existing planning hydraulic model for real-time modeling. Extended Period Simulation (EPS), a major component of an operational model, are hydraulic model simulations in a set time duration that require demand usage patterns and operational controls related to pumps, tanks, and control valves. Establishing when the pumps and/or control valves turn on/off (or throttle) is important since it will allow for accurate prediction once the real-time hydraulic model is running. Ideally, this operational hydraulic model is calibrated to recent field conditions, but this is not necessarily required at this point. The reason being is that the hydraulic model will eventually be linked up to the real-time data during the real-time modeling setup, there will be instant and continuous calibration points to further refine/adjust the model.

Establishing SCADA data access, as this will be the major conduit for data retrieval, is a necessary step in implementing real-time hydraulic models. Typically, this step will be accomplished by linking to a utility's SCADA historian database. A SCADA historian is a software service which stores time-stamped data of SCADA events and alarms in a database which can be queried by a real-time modeling solution. Since this data only provides read-only access there is no security concerns of accessing/controlling SCADA. The real-time model must have the ability to connect to this historian database and access the data in a uniform matter. The real-time modeling platform utilized for the case studies discussed in this paper was Innovyze's IWLive platform which provides the ability to connect to any number of historian databases via native database connections (such as SQL, DB2, Oracle, Pi) as well as OLE DB (Object Linking and Embedding, Database). See Figure 1 for a schematic of the IWLive Architecture.

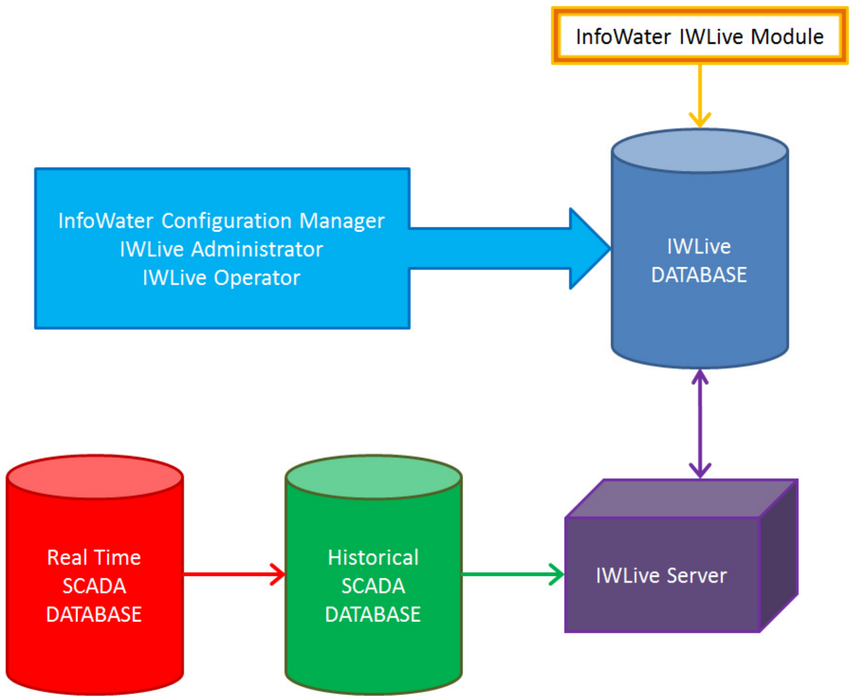

Figure 1 – IWLive Architecture

A necessary step for any real-time modeling platform is configuring SCADA tag mapping to the hydraulic model facility elements. This step can be achieved by developing a list of hydraulic model facilities (pumps, tanks, control valves) by model ID/label and mapping them to the appropriate SCADA tag. Once this list is established, it then can be incorporated within the real-time model platform. SCADA tag mapping is an essential step that is only required to be performed once as long as the SCADA tags do not change.

As many water distribution system operations vary seasonally (and perhaps more frequently) based on demand usage, one needs to establish what demand data sets and related demand patterns needs to be applied within the real –time modeling platform. Demand scaling for different operating conditions can then be applied to the real-time model platform by developing switching criteria for when demand conditions are "switched on". This can be further refined based on available data such as weather forecast data, AMI data, predictive demand data, and related which all can be incorporated within the IWLive platform and used as trigger points to active the specific demand condition.

One of the final steps, which may be considered an ongoing step, is to validate the real-time hydraulic model. IWLive provides a validation results view to identify the deviation from the hydraulic model results when compared to the field results. This provides an opportunity to adjust model parameters to more closely match the field conditions. The common outcome when developing real-time hydraulic models is immediately finding anomalies with either the SCADA data or the hydraulic model data. Accounting for and/or fixing these data challenges (for example, pinned meter readings to inaccurate valve conditions in the model) will further refine the real-time model to better reflect the real-world conditions.

The value of real-time modeling can be increased by configuring easy to understand mapping themes, dashboards, and warning templates to make smarter decisions from. Mapping themes may include contour maps that display key water system performance such as pressure, water quality, available fire flow, etc. Each department in the utility can configure dashboards of the real-time modeling prediction data to quickly assess the status of the system. Warning templates provide the ability to develop alerts where variable thresholds are being exceeded (for example, tanks draining too quickly, sudden drop in pressure, sensor reading stuck at one value, etc.). These alerts can then be sent to the appropriate utility professional via email, social media, etc. so that decisions can be made promptly. Mapping themes, dashboards, reports, and warning templates can all be configured during the implementation stage.

IMPLEMENTATION EXAMPLES

The IWLive software from Innovyze, can be implemented using two of their hydraulic modeling packages, InfoWater and InfoWorks WS. The two examples in this paper were implemented using InfoWorks WS, since their existing hydraulic models were currently in this software platform. The systems had similar, yet different reasons for implementing real-time modeling and process for implementation. These will be briefly described below.

In both implementations, the process of connecting SCADA tags to their model components is the most time consuming process. There is a lot of preparation leading up to this step, which made it move much more smoothly than was anticipated. For example, lists of SCADA tags were reviewed prior to installation to review which were needed, and what the units of their recorded values were. Also, selected data sets were studied to determine if there were any averaging or totalizing occurring that needed to be handled. Finally, a read-only test connection was made to the SCADA database prior to installation to be sure that all permissions and protocols were set up properly. Through this preparatory process, everything was made read for the installation day, and things went very smoothly.

Example 1 – JEA

The Jacksonville Electric Authority (JEA) in Jacksonville, Florida serves an estimated 427,000 electric, 313,000 water and 240,000 sewer customers. In addition, they have an extensive reclaimed water system that has approximately 4,200 customers. Because the reclaimed water system is quite a big smaller than their potable water system, JEA staff decided to install IWLive on the reclaimed system first, for several reasons, before implementing it on the full potable water system. Below are some statistics of the JEA reclaimed system and an overview is shown in Figure 2:

- 10 reclaimed water production facilities
- 31 million gallons per day (MGD) capacity
- 13 MGD average daily flow (ADF)
- 2 storage and re-pump facilities
- 3 production and storage facilities
- 185 miles of pipe

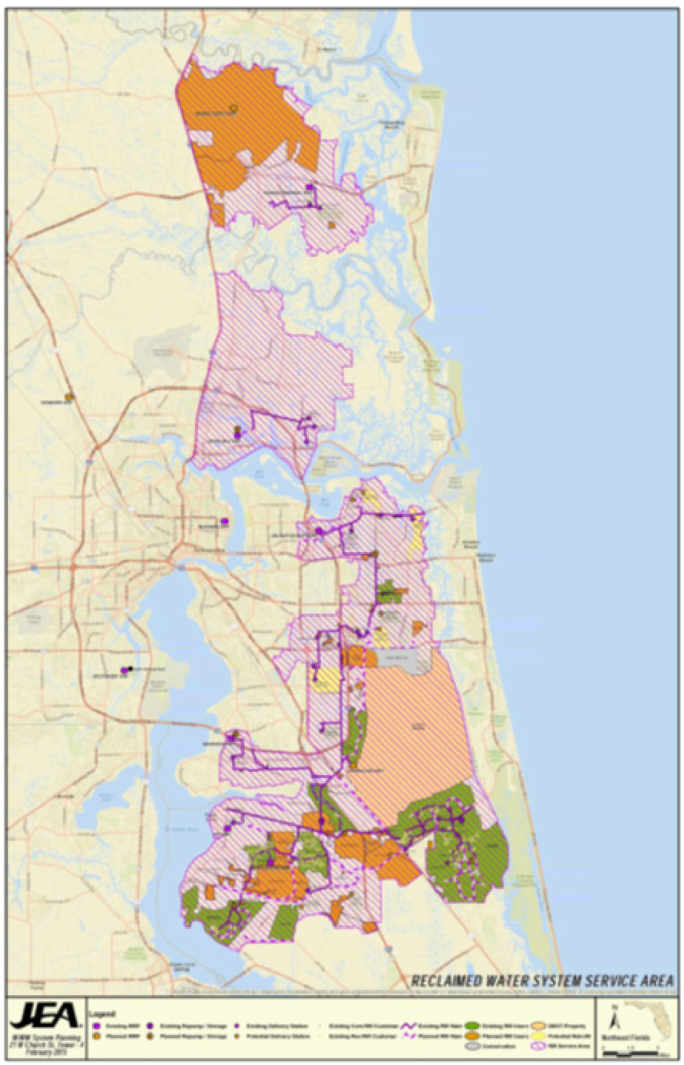

Figure 2 – JEA Reclaimed Water System Service Area

JEA staff decided that since the reclaimed system was smaller than the potable water system, it would install IWLive on that system first, to identify any obstacles and identify solutions. Then the installation on the potable system would go smoother through the lessons learned on the reclaimed system installation. There are many more SCADA tags to link up in the potable system than the reclaimed system. Also, operations staff have multiple responsibilities, and oversee the potable and reclaimed systems, so getting them comfortable and used to the interface on a smaller system first was preferable. *"With all the SCADA data that utilities are recording today—pump runs, system pressures, water quality, plant parameters— it's a shame the data aren't incorporated into calibrating hydraulic models," said Travis Crane, a JEA water/wastewater reliability specialist.*

Example 2 – WVWA

The Western Virginia Water Authority (WVWA) serves drinking water to approximately 158,000 people and sewer service to 120,000 people.in the City of Roanoke, Roanoke County, and Franklin County, Virginia. WVWA staff determined that installing IWLive for their water distribution system would aid in further calibration of the hydraulic model. The following lists the statistics of the WVWA Water System and an overview is shown in Figure 3:

- 55,000 customers
- 3 primary surface water treatment plants
- Several groundwater well locations to supplement supply
- 1,100 miles of water mains

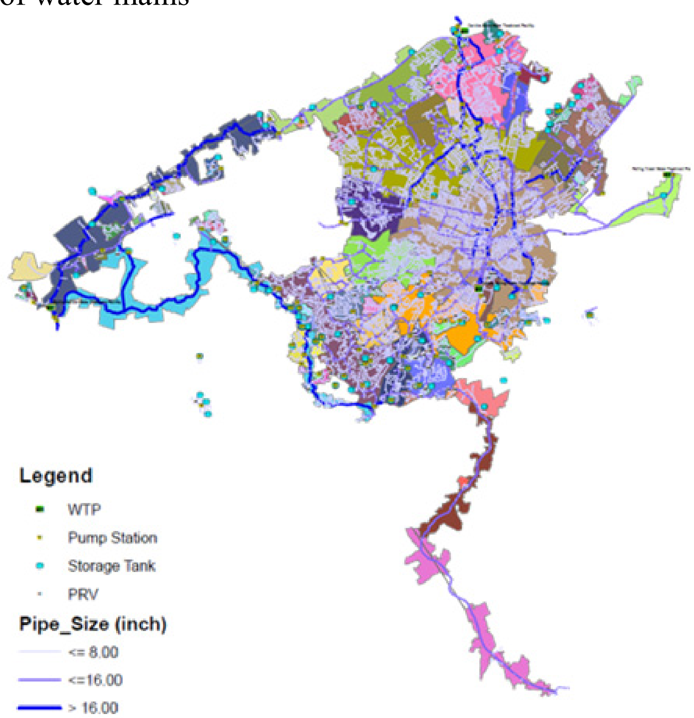

Figure 3 – WVWA Water Distribution System

WVWA staff wanted the IWLive software to help in calibration of the hydraulic model. Upon installation, the comparison of simulated and actual data could be used to calibrate the model. Up to that point, the hydraulic model had been constructed from GIS that was verified in the field, and the model was running. However, field calibration of model simulation results had not been performed. The IWLive real-time modeling solution was identified as a tool to facilitate this process, through the integration of monitoring data from SCADA linked to model nodes. Then, after calibration of the model, it would be introduced to operations staff for their use in operations and forensic analysis.

"Seeing SCADA and model results data together revealed system anomalies the day after initial implementation," said Jim O'Dowd, infrastructure asset manager for the WVWA.

One example of the application of the real-time modeling for calibration is by comparing simulated tank levels from the hydraulic model with actual tank levels from the SCADA. The difference in these two results, is shown in Figure 4 below. In this case, tank levels in the field do not match the tank levels shown in the hydraulic model. The hydraulic modeler can then study the differences and determine changes to make in the model that will better allow the hydraulic model to simulate actual conditions. It should be noted here that it is important to consider the accuracy level of the field equipment and be sure that it has been maintained and calibrated on regular intervals. Possible modifications that could be made to the model include water demand profiles, pump operation controls, and valve settings. In the case of the example in Figure 4, it is likely that a pump was on in the model at the start time, when it should have been off. This could have been due to controls in the model that turn the pump on or off based on the tank level. It appears that the control levels for the pump that affects the level of the tank shown in Figure 4 are on at 17 feet and off at 20 feet. The model results can then be regenerated and compared with the SCADA. Another advantage of having the live SCADA data available and linked is that the simulation can then be compared under different time periods, and the data feed is still available.

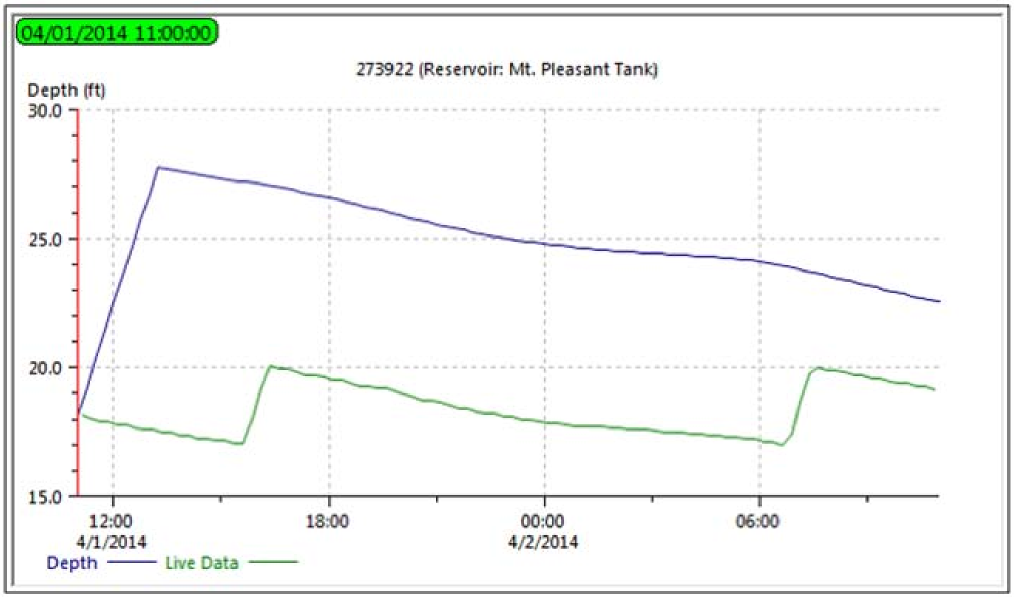

Figure 4 – Example Comparison of Live Data to Model Results for Tank Levels

Another example of using live models to evaluate model calibration of tank levels is shown in Figure 5 below. In this case, the general pattern of tank drain and fill is similar, but the timing is a bit off, and the tank appears to fill at a faster rate in the model than in reality, since the slope of the tank level change is steeper. This could indicate that the water demands around the tank are higher than modeled, or that the pump performance curve is not accurate, or that the system headlosses between the pump and the tank are lower in the model. The ability to visualize the differences and begin to make modifications is another benefit of the real-time modeling.

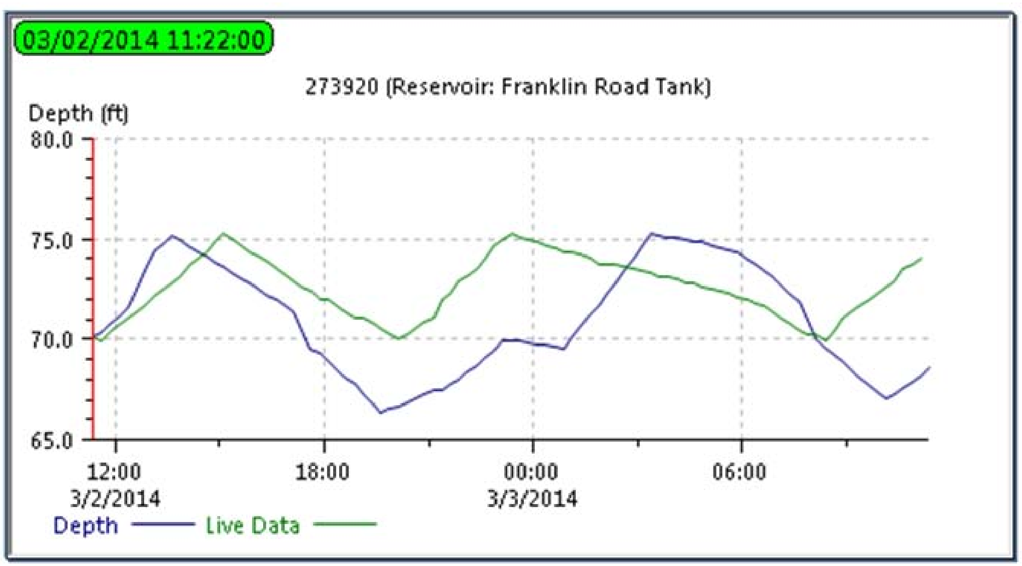

Figure 5 – Another Example Comparison of Live Data to Model Results for Model Calibration

CONCLUSION

There were many opportunities and pitfalls observed during the two examples, and some of those are discussed below. In addition, others will be introduced, even though these two examples didn't experience them. Overall, the installations were a successful and rewarding experience.

Opportunities

In both installations, there were opportunities to educate internal utility staff about the benefits of real-time modeling. Because interface with the SCADA and IT departments was required, to establish the read-only connection to the database, discussions took place about the purpose of the real-time model, and how it could be used. In both cases, there was agreement that the utilities were collecting a lot of data and not really doing that much with it, and the real-time model was a great way to extend the value of existing tools at the utilities.

There are other opportunities for utilities during implementation of a real-time modeling system. One is that they may identify issues or data errors that can be corrected, and may expose operations issues, that were not noticed previously. Another could be that operations data becomes more decentralized to utility staff with varying perspectives on operations.

Pitfalls

There are many potential pitfalls that could impact the success of a real-time model implementation. One is incoming data quality, particularly if there are data gaps or dropouts. There may be need for intermediate data cleaning or scrubbing to prevent bad data being used in the hydraulic model. Another potential pitfall is database integrity and robustness on the SCADA side. If the SCADA database is not set up or used to providing access to data, or the software is old or outdated, there may be a need for some components of the SCADA to be upgraded prior to real-time model implementation. Finally, if the hydraulic model is not fully built or tested for connectivity or the demands are not represented accurately, the real-time model will not be a useful tool. It is recommended that the GIS upon which the model is built be field verified and water demands and asset operations be set up as accurately as possible.

Next Steps

For both of the utility case studies described in this paper, there are lots of next steps. One is the further calibration of their hydraulic models for use in offline simulations like water quality and master planning. Another is operations use of the real-time model to get early warning of system issues or abnormalities that can potentially be avoided. Another is the use of the real-time model for forensic studies of breaks or operational issues. Finally, the ability to predict water demands and operations to optimize quality, quantity and energy and resources has huge potential for both operations. The timeline for implementation of these steps will vary depending on staff availability and model calibration, but both are proceeding.

Summary

The primary focus of this paper was to highlight the benefits of real-time hydraulic modeling and identify the common steps for implementing based on two recent case studies. The benefits of real-time modeling are many and continue to grow as more utilities implement real-time models. By having access to the most recent hydraulic model results tied in with other data silos (such as GIS, AMI, SCADA, etc.) provides the ability to make more informed decisions with confidence. Implementing a real-time system is not a single step but a process that will reveal data challenges along the way. By resolving and/or understanding these data challenges will not only make the real-time model more accurate and improve the confidence level, but it will also provide more return on investment with a utility's SCADA, hydraulic model, and related data systems.

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Development of a Wastewater Pipeline Performance Prediction Model

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Abstract

Performance prediction modeling is a crucial step in assessing the remaining service life of pipelines. Sound infrastructure deterioration models are essential for accurately predicting future conditions that, in turn, are key tools for effective maintenance, repair and rehabilitation decision making. The objective of this research is to develop a wastewater pipeline performance deterioration model for predicting the remaining economic life of wastewater pipe for infrastructure asset management. Under Water Environmental Research Foundation (WERF)'s Strategic Asset Management (SAM) Challenge, there was a planned three-phase development of a pipe deterioration model. Only Phases 1 and 2 were successfully completed under the SAM Challenge. In the Phase 1, the research team identified and developed: life cycle of wastewater pipe; failure modes and mechanisms; consequences of failure; data structures; data collection protocols and methodologies. In the Phase 2, research team developed a standard procedure for rating the performance/condition of wastewater pipes. This paper presents the current progress on Phase 3 research for developing a wastewater pipeline performance deterioration model. This paper demonstrates the research methodology and current progress for update of phase 1 and phase 2 efforts and development of the new phase 3 deterioration prediction model. This research will provide utility managers with a practical and efficient model for the predicting wastewater pipeline performance and estimating end of the remaining life deterioration curve for decision making.

INTRODUCTION

Without efficient investment in the nation's drinking wastewater infrastructure, the environment and public health could be at risk. Performance assessment and prediction are rapidly becoming an increasing part of life-cycle asset management activities in the United States. These models are efficient tools used by infrastructure asset managers to achieve the goal of keeping the performance of wastewater infrastructure at acceptable levels. These models are used to provide decision support to manage this infrastructure and determine where and when resources are needed to be spend.

The long-term funding strategies can be developed based on evaluating what-if scenarios with the use of these performance prediction models. Different idealized strategies for the renewal of assets are shown schematically in Figure 1. Two graphs (A and C) describe theoretical levels of renewal, whereas (B) describes the likely reality of the situation:

- Graph A shows an asset reaching to a minimum acceptable level of service without appropriate renewal. The asset must be renewed or be operationally restricted until necessary renewal works is done.
- Graph C shows an asset that is perfectly constructed, installed, and maintained in its lifecycle. However, very few pipes have such lifecycle, because it is hard to guarantee perfect construction, installation, and maintenance.
- Graph B shows the lifecycle of an asset which is structurally and/or functionally adequate. It has various options for renewal:
  - No action is taken. It then reaches the graph of asset A (red dotted line, strategy a) relatively quickly.
  - The asset is specifically renewed (green arrow) to reach its ideal performance level at its actual age (strategy b).
  - The asset can be further improved (yellow arrow) to the performance level higher than the idea performance level at its age (strategy c).
  - The asset could be repeatedly renewed, as the green saw-tooth graph shows, to maintain an acceptable level of performance over an extended time (strategy d).

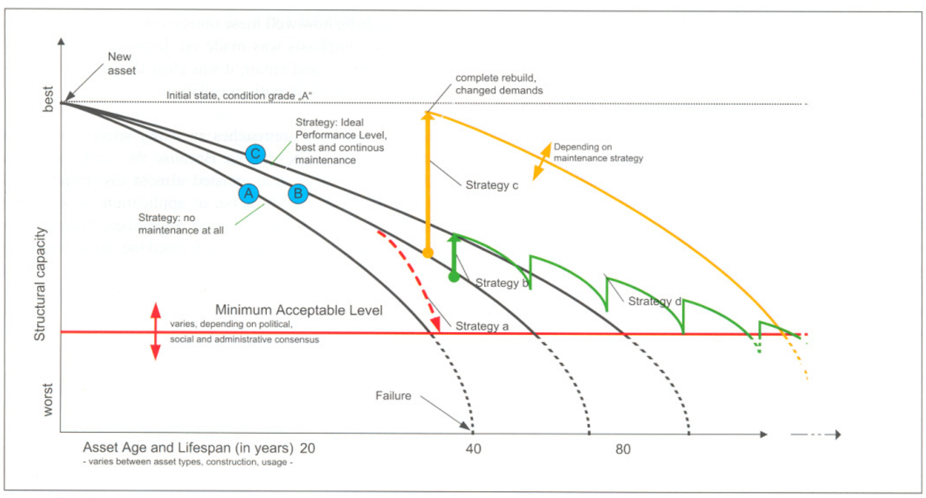

Figure 1. Decision Support with Performance Prediction Models (St. Clair 2014)

LITERATURE REVIEW

Large number of performance prediction models are described in the research literature. Various inputs regarding environmental, structural, functional, and economical factors are evaluated by these model to provide a decision on the management of wastewater pipelines. A short overview of the models are provided in this section. More detailed reviews can be found in Tran et al. (2007), and Ana and Bauwens (2010). Deterioration models for predicting performance of wastewater pipes in the literature can be grouped into four broad categories: Statistical, Probabilistic, and Advanced Mathematical, and Heuristic models.

Statistical Models

Statistical models formalizes the relationship between variables and deterioration in mathematical equations. These models usually rely on historical data collected about the deterioration of the wastewater pipes and tries to put the effect of different variables with correlation approach. The statistical models can be grouped into three categories (linear, exponential, and regression models). Some good examples include; Duchesne et al. (2013), Salman and Salem (2012), Ana and Bauwens (2010), Savic et al. (2009), Chughtai and Zayed (2007), and Wirahadikusumah et al. (2001).

Stochastic or Probabilistic Models

These models assume probabilistic relationship between variables and deterioration. Some good examples for probabilistic models are; stochastic duration models (Mahmoodian et al. 2014), and Markov chain models (Scheidegger et al. 2011, Le Gat 2008, Baik et al. 2006).

Advanced Mathematical Models

These models are generally data driven. Artificial learning algorithms are used to classify the evaluated asset into different categories. Some examples for advanced mathematical models are; fuzzy-based approaches (Angkasuwansiri and Sinha 2014, Kleiner et al. 2007) and neural networks (Tran 2010, Najafi and Kulandaivel 2005).

Heuristic Models

Heuristic models incorporate engineering knowledge rather than data parameters that affect a pipe to determine failure rates. Some examples of these models include; Syachrani et al. (2013), Bai et al. (2008).

Limitations of the Prediction Models in Literature

The literature review indicate that there is no shortage of modeling approaches. Although models evaluated through literature differ in i) mathematical techniques used, ii) the data requirements, and iii) the dataset used for development, following limitations are valid for all:

1. The limits of deterioration prediction capabilities are not in mathematical models or statistical analysis methods, but in lack of accurate and consistent data. The models in literature are created with limited datasets. This limitation causes development without understanding the root causes of deterioration factors and their effect on the deterioration rate.
2. Current models that are in literature and practice are aimed to predict the likelihood of failure (LoF) of the wastewater pipes. LoF models are not useful for the utility managers in tactical and project level decision making since the assets can be interfered long before the failure.
3. The existing models only consider the factors effecting the deterioration in the service state. The distresses that are caused by improper manufacturing, transportation, and installation are not considered in determining the deterioration rates.
4. There are no accuracy assessment for the developed models. The accuracy of the models have not been tested for datasets which have not been used for development.

A verification and validation process needs to be defined in order to test, document, and improve the accuracy of the prediction models.

5. Data on all required parameters may not be available. Prediction models in literature are set to work only with a strict set of parameters and would not give results if some input parameters are missing.

6. To help practitioners on effectively share their decisions with other stakeholders, models need to have various visual reporting capabilities. The model should be developed with GIS capability in order for utilities to run analysis utilizing geospatial data and display results in GIS environment. Additionally, various bar charts, graphs, and visual aids should be developed to visualize the model results.

RESEARCH BACKGROUND

Under Water Environmental Research Foundations' (WERF) Strategic Asset Management (SAM) Challenge, there was a planned three-phase development of a pipe deterioration model. However, only Phases 1 and 2 were successfully completed under the SAM Challenge. In Phase 1, the research team identified and developed: life cycle of wastewater pipe; failure modes and mechanisms; consequences of failure; data structures; data collection protocols and methodologies. In Phase 2, research team developed a standard procedure for rating the performance of wastewater pipes. The third phase, development of the deterioration model has been recently initiated. The three phases of the model development process is presented in Figure 2.

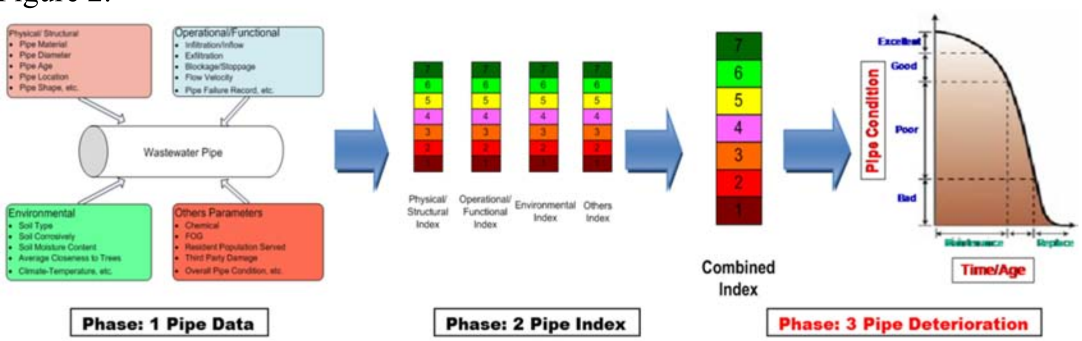

Figure 2. Research Background

Phase 1 Research – Developing Standard Pipe Parameter List (Sinha et al. 2011)

The primary objective Phase 1 research was to develop a set of standard pipe parameter list (data structure). This standard data structure was developed to aid the decision making process in asset management program. In addition, the data structure can be used for developing a condition index, prediction model, prioritizing repair and rehabilitation, prioritizing inspection, planning operation and maintenance, developing a capital improvement program and making. In this phase, the research team investigated the life cycle of wastewater pipeline and identified the causes of pipe failure in different phases including design, manufacture, construction, operation and maintenance, and repair/rehabilitation/replacement. The research team prepared various modes and mechanisms of pipe failure in wastewater infrastructure system as well as identified environmental and societal consequences of the failure. After reviewing all relevant reports and utility databases, the research team has developed a set of standard pipe parameter list (data structure) and pipe data collection methodology. The parameters were divided into five classes based on their characteristics: Physical/Structural, Operational/Functional, Environmental, Financial, and Others.

Phase 2 Research – Development of Performance Index (Angkasuwansiri and 2014a, Angkasuwansiri and Sinha 2014b)

The primary objective of the phase 2 research was development of a performance index for wastewater pipes. This performance index is a performance rating system to evaluate wastewater pipes at the time of inspection. Participating utility data were analyzed to find the statistical significance of each parameter. Some parameters may be missing but, based on a previous study, most of the essential parameters were utilized. 32 out of the 98 parameters defined at Phase 1 research has been used to develop and validate this index. The parameters used in the model are grouped into pipe characteristics, pipe condition (structure), internal and external environment. The performance rating systems evaluate each parameter and combine them mathematically through a weighted summation and a fuzzy interference system that reflects the importance of the various factors.

Phase 3 Research – Development of Performance Prediction Model (Current Progress)

Phase 3 is the current progress for this research project. The main objective of this phase is to develop a prediction performance model for wastewater pipelines. This Phase 3 work will provide utility managers with a practical and efficient technique for the predicting wastewater pipeline performance and estimating end of the remaining life deterioration curve for decision-making. This research will be leveraging previous data standards and performance index established through previous research.

Data is being collected by the support of utilities from various geographical locations. Data collection and analysis effort is supporting understanding the deterioration factors as wells as their single and coupled effects to provide reliable deterioration curves for wastewater pipes. Currently, 32 utilities have provided Memorandum of Understanding (MoU) to the research team to provide data and guidance for the development and piloting of this research phase. Participating utilities are summarized at figure 3.

Figure 3. Participating Utilities for Phase 3 Research.

RESEARCH METHODOLOGY

Development of the proposed deterioration model consists of four major objectives. Objective 1 consists of; determining the list of parameters, define units and ranges for these parameters. Data is been collected with the help of participating utilities to support the later phases. Objective 2 is to update the performance index to assess the performance of gravity wastewater pipelines at the time of inspection. The fuzzy interface algorithm developed for previous research is updated to incorporate these additions. The updated index is being verified by the research team by piloting with participating utilities. Objective 3 is to predict the future performance of the gravity wastewater pipes with using the updated performance index. Updated index will be used for time dependent performance prediction model development. Objective 4 is to integrate the developed model with desktop and online GIS platforms for an effective dissemination. The research methodology is summarized at Table 1. Research team is currently in the progress of updating the performance index and piloting with participating utilities.

Table 1. Research Methodology

Objectives	Objective 1: Data Collection and Parameters	Objective 2. Performance Index	Objective 3. Performance Prediction	Objective 4. Integration and Dissemination
Steps	Update List of Parameters	Update Performance Index	Develop Prediction Model	Integration with GIS
	Data Collection	Pilot Performance Index	Pilot Performance Prediction Model	Integration with PIPEiD

Legend
Completed Tasks
Current Progress

CURRENT PROGRESS

Objective 1. Data Collection and Parameters

<u>Step 1. Update list of Parameters</u>

The list of parameters is revisited and enhanced in order to provide a scientific basis for the development of performance prediction model. 32 utilities U.S. wide has been contacted in order to further evaluate the list of parameters for completeness and accuracy. Additional parameters are defined, the units and the ranges of these parameters are determined. The final list of parameters to develop the performance prediction model is summarized in table 2. Please note that additional parameters to the Phase 2 research is highlighted in gray.

Table 2. List of Phase 3 Parameters

No.	Parameter	Unit	No.	Parameter	Unit
1	Backfill Compaction	Percent	35	Lining Age	Years
2	Backfill Type	Type	36	Lining Material	Type
3	Bedding Condition	Condition	37	Lining pH	pH
4	Bedding Height	Inches	38	Lining Present	Yes/No
5	Bedding Type	Type	39	Lining Type	Type
6	Cathodic Protection	Yes/No	40	Maintenance Frequency	Years

7	Cat. Pro. Design Potential	mV	41	Pipe Age	Years
8	Cat. Pro. Present Potential	mV	42	Pipe Condition	Condition
9	Cleaning Frequency	Years	43	Pipe Depth	Feet
10	Coating Presence	Yes/No	44	Pipe Diameter	Inches
11	Coating Type	Type	45	Pipe Grade	Percent
12	Concrete Encasement	Yes/No	46	Pipe Length	Feet
13	Density of Connections	Con./100ft	47	Pipe Location	Type
14	Dissimilar Materials	Yes/No	48	Pipe Material	Type
15	Distance to WWTP	Miles	49	Pipe Shape	Type
16	Dry Weather Flow	Percent	50	Pipe Slope	Grade
17	Flooding	Yes/No	51	Pipe Surcharging	Yes/No
18	Flow Depth/Diameter	Percent	52	PIPEiD	PIPEiD
19	Flow Velocity	Gal/Min	53	Proximity to Trees	Feet
20	Foreign Anode Bay Distance	Feet	54	Soil Chloride	Percent
21	Frost Penetration	Yes/No	55	Soil Disturbance	Yes/No
22	Ground Cover	Type	56	Soil Moisture	Capacity
23	Groundwater Table	Feet	57	Soil pH	pH
24	H2S	ppm	58	Soil Redox Potential	mV
25	Joint Material	Type	59	Soil Resistivity	ohm cm
26	Joint Material Age	Years	60	Soil Sulfate	mg/l
27	Joint Type	Type	61	Soil Type	Type
28	Lateral Connection Flow Rate	Gal/Min	62	Stray Currents	Yes/No
29	Lat. Con. Height of Drop	Inches	63	Tidal Influence	Yes/No
30	Lateral Connection Location	Angle	64	Type of Cleaning	Type
31	Lateral Connection Size	Inches	65	Wall Thickness	Percent Loss
32	Lateral Connection Slope	Percent	66	Wastewater pH	pH
33	Lateral Connection Type	Type	67	Wastewater Sulfate	mg/l
34	Laying Type	Type	68	Wastewater TSS	Percent

<u>Step 2. Data Collection and Conflation</u>

Data on the list of parameters is being collected from various participating utilities and other data sources. A protocol is followed to collect data from participating utilizes in an effective manner. An initial meeting is held with participating utilities to discuss the list of parameters needed as well as the units and ranges these parameters are recorded. An FTP site is created for utilities to submit the requested data. The initial submitted dataset are evaluated, issues are discussed with a follow up meeting with the participating utilities. The list of utilities already provided data or in the progress on providing the supporting data is summarized at table 3.

Table 3. Data Collection Progress

Utility	Progress
Alexandria Renew Enterprises, Virginia	Receiving Data
American Water, Mount Laurel, NJ	Receiving Data
Boston Water and Sewer Commission, MA	Received GIS and CCTV Inspection Data
City of Baltimore, MD	Received GIS Data
Fairfax County, VA	Receiving Data
Hampton Roads Sanitary District	Receiving Data
Johnson County ,KS	Received GIS and CCTV Inspection Data
Washington Suburban Sanitary Commission	Received GIS and CCTV Inspection Data
Western Virginia Water Authority, VA	Receiving Data

Objective 2. Performance Index

Step 1. Update Performance Index

The performance index developed for previous Phase 2 research is updated for the purpose to be used to predict pipe performance for the future. Additional parameters determined at Objective 1 of the research will be added to the existing modules. Additional modules are added for the failure modes mechanisms omitted for the previous research. Specifically, modules to estimate the lining and joint performance will be added to the current performance index algorithm. Algorithm logic utilizing the fuzzy interface technique will be updated to reflect these additions.

Step 2. Calibration and Verification of the Performance Index

Research team has been piloting the developed performance index with the GIS, defect, and failure data received from Washington Suburban Sanitary Commission (WSSC). These records contain data for 154,675 pipe segments. 112 of these pipe was randomly selected to be evaluated. Extracted data from utility records are summarized in Table 4.

Table 4. Parameters Extracted from Utility Data

Parameter	Source
Pipe Age	Geodatabase
Pipe Condition	CCTV Inspection Data
Pipe Depth	CCTV Inspection Data
Pipe Diameter	CCTV Inspection Data
Pipe Length	CCTV Inspection Data
Pipe Location	Geodatabase
Pipe Slope	CCTV Inspection Data
Pipe Surcharging	Failure Reports
Lining Presence	CCTV Inspection Data
Lining Type	CCTV Inspection Data
Flow Depth/Diameter	CCTV Inspection Data
Density of Connections	CCTV Inspection Data
Flow Velocity	Geodatabase

Piloting Results Discussion

A focused dataset with 112 pipe segments was used to pilot the pipe performance index. The results differences between the PACP grades and index outputs range between 0 and 3. Table 5 summarizes the overall results for the focused dataset.

Table 5. Final Piloting Results

Total Number of Segments	Segments with 0 difference	Segments with 1 difference	Segments with 2 Difference	Segments with 3 Difference
112	31	55	22	4
100%	27.7%	49.1%	19.6%	3.6%

Results with 0 or 1 Difference

Results for the pipe segments where there is 0 or 1 difference between the PACP grade and the index output indicate the pipes with the desirable parameters (low range) tend to give results closer to the PACP grade. Additionally, pipes with PACP grade of 5 tend to give the same result for the index. Tables 6 and 7 summarize pipe segments with 0 and 1 differences.

Table 6. Sample Segments with 0 Difference

Significant Parameter	PIPEiD	Index	PACP	Difference
No Load	68	3	3	0
Small diameter	484	3	3	0
Newer Pipes	2833	3	3	0
Low Velocity Pipe	5676	3	3	0
Newer Pipes	5895	0	0	0
Short Pipes	6786	0	0	0

Table 7. Sample Segments with 1 Difference

	PIPEiD	Model	PACP	Difference
Under Highway	2	1	0	1
Pipes with High Slopes	65	3	2	1
Low Capacity	67	1	0	1
Pipes with high density connections	733	4	3	1
Old Pipes	1142	3	2	1
Shallow Pipes	1230	1	0	1
Large Diameter	1389	3	2	1
High Velocity Pipe	1424	1	0	1
Under highway	2165	3	2	1
Pipes with High Slopes	2584	3	2	1
Newer Pipes	2835	3	2	1

Results with 2 Difference

There are 22 (19.6%) pipe segments where there is 2 difference between the PACP grade and the index output. Table 8 summarizes sections with 2 difference.

Table 8. Segments with 2 Difference

PIPEiD	Model	PACP	Difference	Max Result Module
540	2	0	2	blockage
861	2	0	2	blockage
1170	2	0	2	blockage
1301	2	0	2	blockage
1390	2	0	2	integrity
2535	2	0	2	surface
3151	2	0	2	blockage
4355	2	0	2	blockage
4630	2	0	2	blockage
4658	2	0	2	blockage
5092	2	0	2	blockage
5540	2	0	2	blockage
8193	2	0	2	blockage
9339	2	0	2	surface

1143	3	1	2	integrity
3889	3	1	2	blockage
9269	3	1	2	integrity
9275	3	1	2	integrity
85	4	2	2	integrity
3023	4	2	2	capacity
4652	4	2	2	capacity
9693	4	2	2	capacity

Pipe Segment #1390

Table 9. Pipe Segment #1390

Parameter	Value	Parameter	Value
PIPEiD	1390	Pipe Slope	0.73913
Pipe Age	42	Pipe Surcharging	0
Pipe Condition	0	Pipe Grade	0.73913
Pipe Depth	0.338417	Lining Present	-1
Pipe Diameter	8	Lining Type	0
Pipe Length	264.5	Flow Depth/Diameter	0.1
Pipe Location	4	Flow Velocity	0.554
		Density of Connections	2

Discussion: Although there are no defect noted by the CCTV inspection, the pipe is located under a major highway and pipe depth is shallow. These parameters indicate that there is high amount of dynamic loading on the pipe which makes it prone to integrity issues.

Results with 3 Difference

Table 10. Segments with 3 Difference

PIPEiD	Model	PACP	Difference	Max Result Module
381	3	0	3	capacity
2056	3	0	3	capacity
5554	3	0	3	capacity
9593	3	0	3	capacity

Pipe Segment # 381

Table 11. Pipe Segment #381

Parameter	Value	Parameter	Value
PIPEiD	381	Pipe Slope	5.59
Pipe Age	18	Pipe Surcharging	0
Pipe Condition	0	Pipe Grade	5.59
Pipe Depth	2.90025	Lining Present	-1
Pipe Diameter	8	Lining Type	0
Pipe Length	112.5	Flow Depth/Diameter	0.95
Pipe Location	4	Flow Velocity	1.853
		Density of Connections	0

Discussion: Although the PACP grade for the pipe is 0, this specific segment of pipe is operating in full (95%) capacity level. This is a proof that the pipe has capacity issues.

FUTURE RESEARCH

Objective 3. Performance Prediction

Step 1. Develop Mathematical Model – Preliminary Results

The data received from WSSC was utilized to illustrate the implementation of the performance prediction model and represent preliminary results. For the preliminary results, Gravity concrete pipes with diameter less than 15" located in specific area called Broad Creek Basin were randomly selected. In order to develop the preliminary deterioration curve, the performance transition probability matrices were calculated. These transition probability matrices were then used to determine the expected performance at a given time with the expected value method. Figure 4 summarize the preliminary results for the performance prediction model.

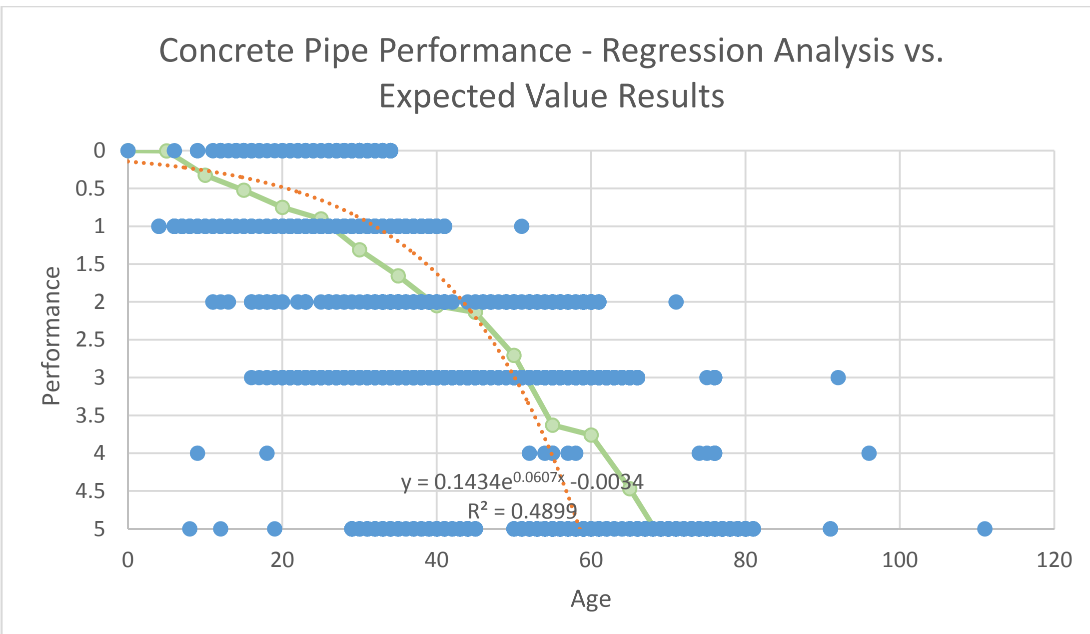

Figure 4. Preliminary Results for Performance Prediction.

Based on the preliminary performance prediction curves, it can estimated that the performance value of gravity concrete pipes smaller than 15” diameter located in Broad Creek Basin will move to "5" (failed) at the age of 70 years old, assuming that there will be no rehabilitation work performed (run to failure). Please note that the results of the performance prediction model is created without historical panel data and it depends directly on the performance index results. The R square value is 48.99% which is relatively low (out of 100%) suggesting that the correlation of the data is weak.

Step 2. Pilot and Validate Performance Model with Participating Utilities

The piloting and validation of the performance prediction model brings new challenges because of the time dependency of the performance prediction. Usually, inspections are conducted ad-hoc in random times through the pipe life. Furthermore, utilities have extensive failure databases which can be used in order to be used as a ground truth for the end of pipes service life. Lack of panel historical data limits of the validation which can be conducted. Although these limitations exist, there are partial historical data available for the pipes which are in service are available and piloting will be conducted with this partial historical data. Tests sites to conduct blind tests will be determined with the help of utilities according to the data availability and willingness of the utility to further investigate the selected sites. Pipe samples will be used in two types of tests.

Objective 4. Integration and Dissemination

Step 1. Integration with GIS (Desktop)

Updated performance index and newly developed prediction model will have the capabilities to provide visual outputs such as graphs and charts. These visual outputs will be used by practitioner to effectively evaluate and share with other stakeholders. Index and the prediction model will be integrated with the Geographical Information System (GIS). Develop algorithms will be able to process the location of the pipes and display the outputs on the GIS system (ArcGIS 10.2).

Step 2. Integration of Wastewater Pipeline Model with Pipeline Infrastructure Database (PIPEiD)

PIPEiD (Pipeline Infrastructure Database) is an interoperable platform for the development, implementation, and benchmarking of models, to enable accurate quantitative analysis. Proposed model will be able to run as a standalone application at user desktops as well as the PIPEiD platform utilizing the accumulated data. Implementation of the proposed model at the PIPEiD platform will provide ease of use and eliminates interoperability issues.

CONCLUSIONS

Accurate prediction of wastewater pipe performance plays an essential role in asset management and capital improvement planning. This paper discuss the development of a performance prediction model for wastewater pipes. Leveraging previous research for standardized data and performance index, this research will provide utility managers with a practical and efficient technique for the predicting wastewater pipeline performance and estimating end of the remaining life deterioration curve for decision-making. A list of parameters is established and the historical data for these parameters will be collected from the participating utilities. Collected data will support understanding the deterioration factors as wells as their single and coupled effects to provide reliable deterioration curves for wastewater pipes. Relationships between the performance state and the deterioration factors will be investigated. Established plan to pilot and validate with participating utilities will ensure the accuracy and the acceptance of the developed model. Additionally, integration with the PIPEiD Platform will provide effective dissemination and utilization of the prediction model by utilizes nationwide.

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Pressure and Transient Monitoring of Water Transmission Pipelines and Wastewater Force Mains

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Abstract

The operation of water transmission systems and recurring nature of pump cycling at wastewater pumping stations contributes to pipeline fatigue and stress and can impact a pipeline's structural integrity and useful life. Pressure and transient monitoring of water transmission pipelines and wastewater force mains provides information that can be useful to a Utility's system operations and Asset Management. When compared to design values, and historical system records, analysis of pressure and transient data can provide additional quantified information to help determine the risk or likelihood of failure associated with a pipeline or force main to be applied to an overall asset management plan. This quantified operational information can also be used to support implementation of or changes to standard operation procedures (SOPs) and maintenance and repair programs. This paper will include a brief overview of various available pressure logging/monitoring technologies and how the technologies have been implemented in several different applications. Up to five case studies will be presented (water transmission pipeline & wastewater force mains) and will include the implementation, analysis, and results of the associated pressure and transient monitoring, including impacts to the associated pipeline and system, and recommendations.

INTRODUCTION

Transient pressures are well understood theoretically and are considered in standards and the design and operation of water and wastewater systems. However, they are not often detected and measured in the field. This makes it difficult for Utilities to address the ongoing and changing impact of surge on their systems, particularly as assets age and deteriorate and systems change from the original design conditions.

The majority of water and wastewater systems do not have integrated transient capable detection instrumentation. Measurement of the actual loading an asset is subject to, including transient pressures, is necessary for accurate structural evaluation and an integral part of comprehensive condition assessment. Detection and measurement of transient events is also a necessary step to making recommendations for their mitigation, reducing loading to prolong the life of assets. Transient pressure monitoring provides a complete picture of how a system is operating and the actual loads assets are subject to. Transient capable pressure monitoring needs to be a part of every condition assessment project and a Utility's overall asset management strategy.

BACKGROUND

A Water Research Foundation survey [1] of 36 small, medium, and large Utilities revealed the following about Utility system hydraulics:

- 95% of the utilities surveyed have a requirement of at least 20 psi during fire flow (of the remaining 5%, half required 0 pounds per square inch (psi) and the other half 30 psi).
- 68% require a minimum of 20 psi during emergency conditions (i.e. main break).
- 21% require a minimum of 0 psi during emergency conditions (i.e. main break).
- 5.3% have no requirement for minimum pressure during emergency conditions.
- 65% have *no requirement* for maximum system pressure.
    - The remaining Utilities vary widely between 65 psi and 320 psi for maximum allowable system pressure.
- 13% utilize targeted pressure monitoring; the remaining 87% utilized convenient available locations (pump station, storage tank, treatment plant).
- Only 10% of Utilities surveyed utilize a pressure data recording frequency [sample rate] less than 1 minute.

These survey results provide evidence that many Utilities do not have active pressure management and do not utilize remote (non-facility) transient pressure monitoring. The final point provides evidence that while many Utilities may utilize conventional pressure monitoring at their facilities, most Utilities do not have transient capable pressure monitoring or detection instrumentation in place. Without this instrumentation in place detection of the actual extent of pressures affecting a system over time are not know.

Local Monitoring vs. Remote Logging vs. Remote Monitoring. Conventional water network data collection typically occurs on a local level, where systems are installed at existing facilities such as pump stations, tanks / reservoirs, and treatment plants. These installations are supplied with direct power and communications connections and often transmit directly to (and receive from) an existing Supervisory Control and Data Acquisition (SCADA) system. Remote data collection presents several challenges compared to local monitoring: power is limited by battery life, there is no standard data retrieval or communication arrangement, wireless communications require an existing wireless network and an adequate signal.

Until recently remote data collection has been through manual retrieval / download from remote loggers that collect data and store it on board. With the widespread use of cellular technology, remote wireless communication has become more accessible. This wireless accessibility, along with advances in battery technology and data communications, has made distributed remote monitoring feasible.

Transient vs. Conventional Pressure Monitoring. Transient pressure monitoring can be differentiated from conventional pressure monitoring by the rate at which the pressure is being sampled. Conventional pressure monitoring systems are common in water networks. Most SCADA linked conventional pressure monitoring systems sample at a rate of 1 sample / minute (0.016 Hz) or less.

High sample rate transient pressure monitoring is relatively new for water network instrumentation. As such, there is no standard for a minimum transient sampling rate.

The theoretical wave speed of a transient can be calculated by the equation [2]:

$$c = \frac{1}{\sqrt{\rho\left(\frac{C_1 D}{tE} + \frac{1}{K}\right)}}$$

c = Acoustic wave speed (m/s)
E = Young's modulus of pipe material (N/m^2)
K = Bulk modulus of fluid (N/m^2)
ρ = Fluid density (kg/m^3)
D = Pipe diameter (m)
t = Pipe wall thickness (m)
C_1 = Constant depending on pipe anchorage

The main variables that affect the wave speed of a potential transient are water temperature (density), pipe material, pipe diameter, and pipe wall thickness. Theoretical acoustic wave speed values [3] for Prestressed Concrete Cylinder Pipe (PCCP) generally range from 1,198 meters per second (m/s) for 406.4 millimeter (mm) (16-inch) Lined Cylinder Pipe (LCP) at 1,379 kilopascal (kPa) (200 psi) to 878 m/s for 1828.8 mm (72-inch) Embedded Cylinder Pipe (ECP) at 689 kPa (100 psi). The theoretical wave speed continues to decrease for larger diameter pipe and lower operating pressures. At the maximum theoretical wave speed of 1,198 m/s a transient would pass a 76.2 mm diameter tap in 0.0000625 seconds. The minimum required sampling frequency for detection is commonly defined as twice the frequency of the event to be detected. The minimum required sample rate to detect a 0.0000625 second duration event is 32,000 Hz.

Transient data collected to date suggests that transients in water networks do not approach this very high theoretical wave speed. It is also possible that there is some other mechanism at work that acts to slow the translation of the high wave speed transient at the pressure sensor. Figure 1, below, is transient pressure data from a 304.8 mm (12-inch) ductile iron force main with a theoretical wave speed of 1,198 m/s. The x-axis spans a period of 2 minutes with a grid spacing of 1 second. The wavelength of the captured transient pressure wave is approximately 5 seconds. This equates to a frequency of 0.2 Hz.

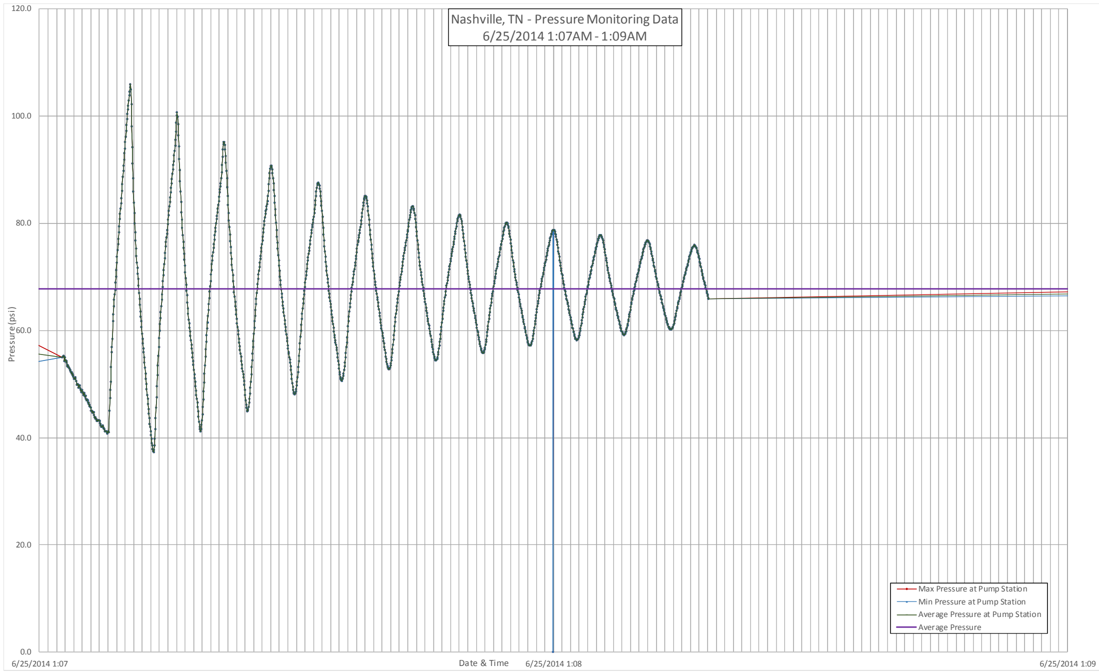

Figure 1. Transient Data, 12-Inch DIP Force Main.

For the example in Figure 1 the minimum required sampling frequency to detect the event would be 0.4 Hz. To capture an accurate representation of the detected event, the sampling frequency needs to be higher. The data in Figure 1 was captured at a sample rate of 20 Hz; 100 times higher than the frequency of the event detected. Based on the transient pressure data monitoring across various water systems, including the Washington Suburban Sanitary Commission (WSSC), a minimum sample rate of 20 Hz reveals transient events with frequencies significantly lower than the maximum detection limit of these sensors in pipelines greater than 304.8 mm.

Available Technologies. This paper focuses on remote transient pressure monitors with wireless communication capability to provide near-real-time data. Available remote transient pressure monitors include:

HWM Water Ltd – GPRS Transient Logger
Smart Water Services LLC – Wireless RTU (ECO Series / PRO Series)
GCR Tech – GPRS TRITON Pressure Transient Logger
Telog Instruments - PR-32iv
Syrinix – TransientMinder

Pressure Monitoring in the Water and Wastewater Industry.
State-of-the-Industry – Varies from no pressure monitoring to well established SCADA based conventional pressure monitoring at system facilities (pump stations, storage tanks), low sample rate (non-transient).

Best Practice – Optimized pressure monitoring and pressure management as defined by the EPA. Pressure monitoring at far reaches of the system, maximum and minimum pressure points (critical points) in multiple pressure zones, low sample rate (non-transient).

State-of-the-Art – Real-time high rate transient capable pressure monitoring at remote locations throughout the network.

MATERIALS AND METHODS

Equipment Specifications. Several transient pressure technologies have been used on water transmission mains and wastewater force mains to detect and quantify transients. A summary of the specifications for these units is included in Table 1.

Table 1. Transient Pressure Monitor Specifications.

Specification	Telog – LPR-31i	Telog - PR-32i [HPR-32i]	Syrinix - TransientMinder
Connection	¼" NPT – direct connection	¼" NPT direct connection [3.5" NHT]	21KA air hose connection indirect connection
Sensor Enclosure Rating	IP68 (dust tight, suitable for immersion in liquids beyond 1m)	IP68 (dust tight, suitable for immersion in liquids beyond 1m)	Integrated in transmitter housing
Transmitter Enclosure Rating	N/A	IP66 (dust-tight, suitable for powerful water jet projection)	IP68 (dust tight, suitable for immersion in liquids beyond 1m)
Cellular Communication	N/A	Integrated (no SIM card)	SIM card required, provided by customer
Cellular Provider	N/A	Verizon	AT&T
Battery Life	1-5 years	1-5 years	1-3 years
Max. Sample Rate	20 Hz	32 Hz	128 Hz
Antenna	N/A	External, direct-buried, stainless steel and epoxy construction [Integrated]	External, plastic construction
Accuracy	0.25% of full scale, temperature compensated	Not published, similar to LPR-31i	Not published
Range	-15 to 300 psi	-15 to 300 psi	20 Bar absolute

Equipment Installation. The transient pressure monitors were connected to the pipelines via a maximum 76.2 mm (3 inch) tap and then a reduced 19.05 mm (3/4 inch) tee tapped into the approximately 304.8 mm (1 foot) long 76.2 mm (3 inch) diameter spool piece. A ball valve, vent/drain, and analog pressure gauge connection are also provided. The Telog LPR-31i and PR-32iv pressure sensors connect directly to the reduced 19.05 mm tap. The Telog HPR-32iv connects directly to a charged fire hydrant via a 3.5” National Hydrant Thread (NHT) connection. The Syrinix TransientMinder pressure sensor is housed in the transmitter body and connects to the reduced 19.05 mm (3/4 inch) tap via a 2,000 mm long, 19 mm diameter air hose.

Figure 2. Telog PR-32iv Wireless Transient Pressure Monitor Installation.

Equipment Operation. The available transient pressure monitoring units operate in a similar manner. The units continuously monitor pressure at a high sample rate, while only recording data every few minutes under normal operating conditions (based on user defined parameters). When a transient is detected, data is recorded continuously at the high sample rate. The settings for triggering the high recording rate are user programmable. The parameters for the Telog transient trigger setting are change in pressure and change in time. The Syrinix unit uses a proprietary unitless sensitivity setting. The conventional operating pressure recording settings and transient pressure recording settings are summarized along with the results from each case study presented.

RESULTS

Case Study 1 – 304.8 mm (12-Inch) Ductile Iron (DIP) Force Main. Two transient pressure loggers were installed on a 304.8 mm (12-inch) wastewater force main (FM) in Nashville, TN to determine the discharge condition of the force main and detect and quantify potential surge pressures. The transient pressure logger settings are summarized in Table 2.

Table 2. Nashville, TN FM Pressure Logger Settings.

Parameter	Setting
Pressure Sensor Range	-15 to 300 psi
Sample Rate	20 Hz (50 mSec interval)
Standard Recording Interval	2 min (min., avg., max.)
Transient Trigger ΔP	20 psi
Transient Trigger Δt	2.5 sec
Pre-Impulse Recording Period	5 sec
Post-Impulse Recording Period	25 sec

The downstream pressure logger revealed that the force main was transitioning to gravity prior to the gravity transition manhole. The upstream pressure logger, installed at the pump station, indicated that the average pressure in the force main ranges from 470 kPa (68.2 psi) at static pressure to 487 kPa (70.7 psi) with the pump operating as illustrated in green in Figure 4. The maximum and minimum values recorded over each 2-minute interval at the high sample rate revealed peak and low pressures associated with each pump cycle that range between 772 kPa (112 psi) and -1.4 kPa (-0.2 psi) as illustrated in red and blue in Figure 3.

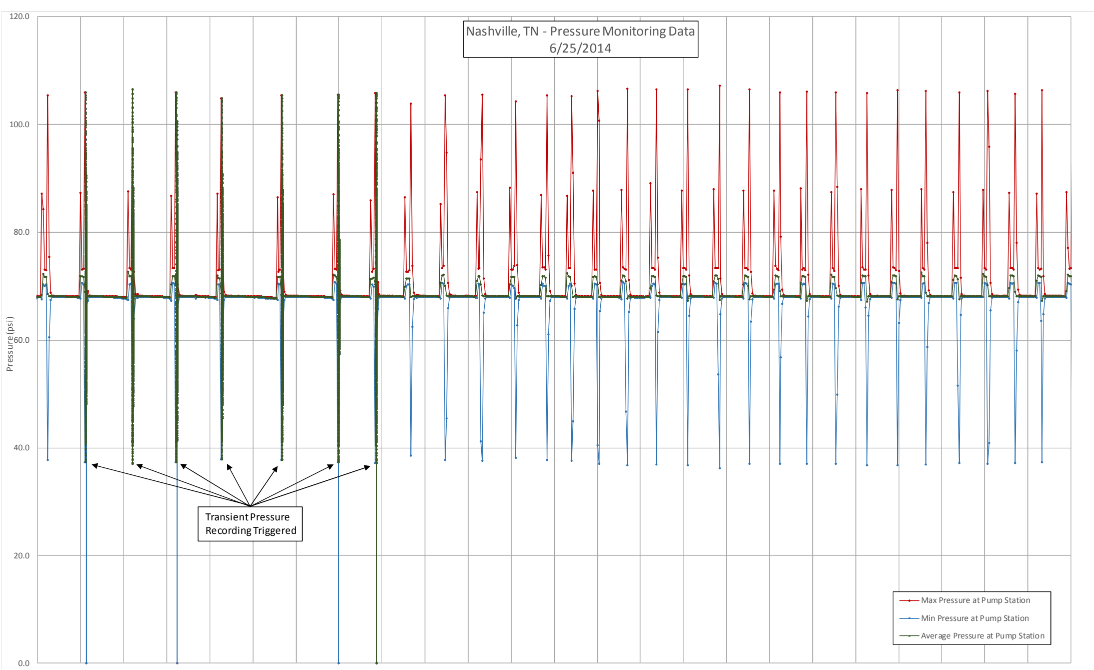

Figure 3. Nashville, TN 304.8 mm (12-Inch) DIP Force Main, Peak and Low Pressures.

A detailed view of a single recorded transient event is shown in Figure 1. The wavelength of these transient events is 5 seconds.

The average pump cycle frequency of 1.25 pump cycles per hour, as determined from the pressure data, is within suggested guidelines for design of pumps and motors at wastewater pump stations [4]. The pressure data also revealed that recurring transient pressure events occur during pump shutdown. The recorded pump start/pump stop cycle frequency for the Force Main is 2.5 cycles per hour and the range is approximately 138 kPa (20 psi) for half of the cycles (pump start) and 483 kPa (70 psi) for the other half of the cycles (pump stop). Inclusion of the pressure fluctuations within each transient event, as shown in Figure 1, increases the pressure cycle frequency experienced by the force main from 2.5 pressure cycles per hour to approximately 18-20 pressure cycles per hour. The amplitude of the pressure cycles ranges from approximately 138 kPa (20 psi) to 483 kPa (70 psi). The design operating pressure of new 304.8 mm (12-inch) DIP is 2,413 kPa (350 psi). While the maximum recorded and regularly recurring peak pressures of 772 kPa (112 psi) do not approach the new DIP design pressure for standard 350 Pressure Class, or exceed the design surge allowance, the actual measured maximum pressures and fatigue due to cyclic loading, in conjunction with other deterioration modes, such as internal corrosion due to hydrogen sulfide, may be a concern at the Force Main. A structural fatigue model may be developed to quantify the impact of the actual peak pressures and observed pressure cycling on the Force Main in conjunction with various amounts of possible wall loss. Surge protection improvements or maintenance of existing surge mitigation equipment may also reduce or eliminate the recurring pressure transient that occurs during pump shutdown and extend the life of the asset.

Case Study 2 – 203.2 mm (8-Inch) PVC Force Main. A transient pressure logger was installed on a 203.2 mm (8-inch) wastewater force main (FM) in California as part of a pipeline assessment to detect and quantify potential surge pressures. The transient pressure logger settings are summarized in Table 3.

Table 3. FM Pressure Logger Settings.

Parameter	Setting
Pressure Sensor Range	-15 to 300 psi
Sample Rate	20 Hz (50 mSec interval)
Standard Recording Interval	1 min (min., avg., max.)
Transient Trigger ΔP	30 psi
Transient Trigger Δt	2.5 sec
Pre-Impulse Recording Period	5 sec
Post-Impulse Recording Period	25 sec

The pressure logger, installed at the pump station, indicated that while the average pressure in the force main ranges from 16 kPa (2.3 psi) at static pressure to 50 kPa (7.2 psi) with the pump operating the peak and low pressures associated with each pump cycle range from 205 kPa (29.8 psi) to -41 kPa (-6.0 psi). Analysis of the pressure data also revealed approximately 4.1 pump cycles occur per hour. Pressure peaks and lows on pump start and pump stop resulting in two pressure cycles for every one pump cycle, or a total of 6.1 pressure cycles per hour with a range of 247 kPa (35.8 psi).

Structural fatigue modeling of the quantified pressure variation and cycling rate was performed using the Vinson Method as well as Moser's Method. Based on the recorded pressure cycle and amplitude the expected fatigue life of the force main as new is approximately 200 years. This estimated life expectancy is based solely on the recurring transient pressure and may be further reduced due to other structural impacts on the pipeline such as damage caused during installation, corrosive environment, bedding, or backfill conditions.

Case Study 3 – Cobb County, GA, 1,067 mm (42-Inch) Transmission Main. A transient pressure logger was installed on a 1,067 mm (42-inch) PCCP water transmission main (TM) in Georgia as part of a pipeline assessment to detect and quantify potential surge pressures. The transient pressure logger settings are summarized in Table 4.

Table 4. Cobb County, GA TM Pressure Logger Settings.

Parameter	Setting
Pressure Sensor Range	-15 to 300 psi
Sample Rate	20 Hz (50 mSec interval)
Standard Recording Interval	2 min (min., max.)
Transient Trigger ΔP	10 psi
Transient Trigger Δt	1 sec
Pre-Impulse Recording Period	5 sec
Post-Impulse Recording Period	10 sec

The pressure logger, installed at the pump station, recorded an average operating pressure of 1,089 kPa (158 psi). The pressure data reveals approximately 1 pressure cycle occur per day with an operating pressure range of 1,069 kPa (155 psi) to 1,138 kPa (165 psi); this is consistent with the expected diurnal pattern for a potable water system. Approximately 60 transient events were also recorded during the 55 day logging period. The maximum pressure recorded was 1,296 kPa (188 psi) and the minimum pressure was 662 kPa (96 psi), these extreme pressures occurred during a single transient event on March 19, 2014, shown in Figure 4.

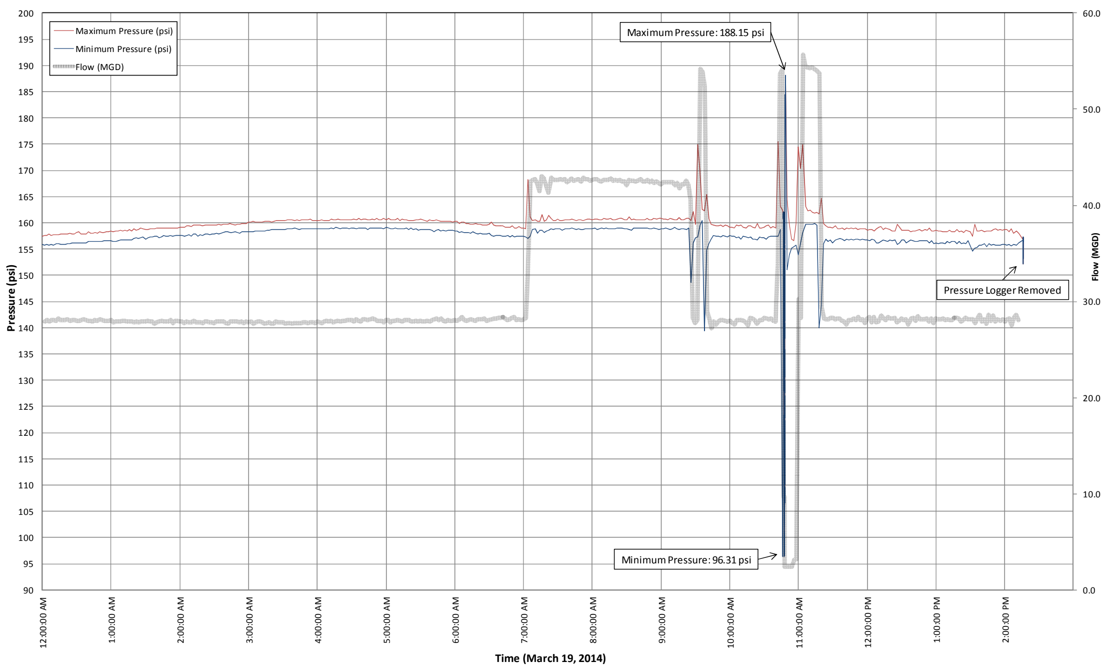

Figure 4. Cobb County, GA 1,067 mm (42-Inch) Transmission Main, Transient Event.

Pump testing was being conducted on March 19, 2014 and this transient event coincided with the simultaneous shutdown of two operating pumps as confirmed by the pump station operational and flow SCADA records. Flow is shown as the thick grey line on the secondary y-axis in Figure 5, and the raw SCADA pressure and operational data, recorded at 1-minute intervals, is shown in Figure 5.

	HSPS East and West Flow Total (MG) (F_CV)	High Service Pump Station Header Pressur (F_CV)	High Service Pump No. 1 Running St (F_CV)	High Service Pump No. 2 Running St (F_CV)	High Service Pump No. 3 Running St (F_CV)	High Service Pump No. 4 Running St (F_CV)	High Service Pump No. 5 Running St (F_CV)
19-Mar-14 10:44:00	53.6	162.8	1	0	Bad	1	0
19-Mar-14 10:45:00	54.1	161.7	1	0	Bad	1	0
19-Mar-14 10:46:00	54.1	162.2	1	0	Bad	1	0
19-Mar-14 10:47:00	27.1	128.2	0	0	Bad	0	0
19-Mar-14 10:48:00	9.7	166.1	0	0	Bad	0	0
19-Mar-14 10:49:00	3.8	156.5	0	0	Bad	0	0
19-Mar-14 10:50:00	2.4	153.7	0	0	Bad	0	0

Figure 5. Cobb County, GA 1,067 mm (42-Inch) Transmission Main, SCADA.

The pump station SCADA data indicates a minimum pressure of 884 kPa (128.2 psi) and a maximum pressure of 1,145 kPa (166.1 psi) at the 1-minute intervals bracketing the transient event. The amplitude of the pressure swing in the SCADA records does not present the full extent of the transient pressure as recorded by the transient pressure logger, shown in Figure 5. Several other transient events were also recorded on March 19, 2014 that correlate with pump operation, summarized in Table 5.

Table 5. Cobb County, GA 3/19/2014 Transient Events.

Time	Transient Pressure Event	PS Operation
7:02 AM	Spike to 1,207 kPa (175 psi)	Pump No. 2 turns ON
9:26 AM	Drop to 1,027 kPa (149 psi)	Pump No. 2 turns OFF
9:33 AM	Spike to 1,207 kPa (175 psi)	Pump No. 1 turns ON
9:38 AM	Drop to 965 kPa (140 psi)	Pump No. 1 turns OFF

The transient pressure monitoring confirmed the occurrence of recurring transient pressure events at the 1,067 mm (42-Inch) Transmission Main. The amplitude of a typical transient event is 207-276 kPa (30-40 psi) and the frequency is 7 to 8 times per week. These moderate transient pressure events correlate with operational changes (pump start / pump stop) based on available SCADA information.

One severe transient pressure event was also detected with a pressure deviation of 689 kPa (100 psi) and duration of approximately 1 minute. This event correlates with the simultaneous shut-off of two operating pumps based on SCADA information. The SCADA 'High Service Pump Station Header Pressure', recorded at 1-minute intervals, does not provide enough resolution to capture the true extent or severity of transient pressure events that occur at the Pump Station. Hydraulic modeling indicates that transient pressure events often have the greatest impacts where pressure waves can be reflected, and where gas can accumulate and column separation may occur. Further investigation of bends and high points in the pipeline was recommended to determine if any damage was observed at these concentration points.

PCCP is a composite material, the main structural component of which is the prestressing wire. Inspection techniques have been developed that can estimate the number of broken prestressing wire wraps in a given pipe. Structural finite element analysis (FEA) may be performed based on the detailed design specifications of a given PCCP design, the pipe loading conditions, including earth cover and internal pressure, and the estimated number of broken prestressing wire wraps of a pipe to evaluate the structural performance of the pipe. Actual loading may be different than original design values. Transient pressure monitoring allows the quantification of internal pressure for evaluation as part of the structural model. The design pressure of the transmission main is 1,724 kPa (250 psi), however, in conjunction with prestressing wire breakage, the theoretical yield strength of the pipe is reduced, as shown in Figure 6.

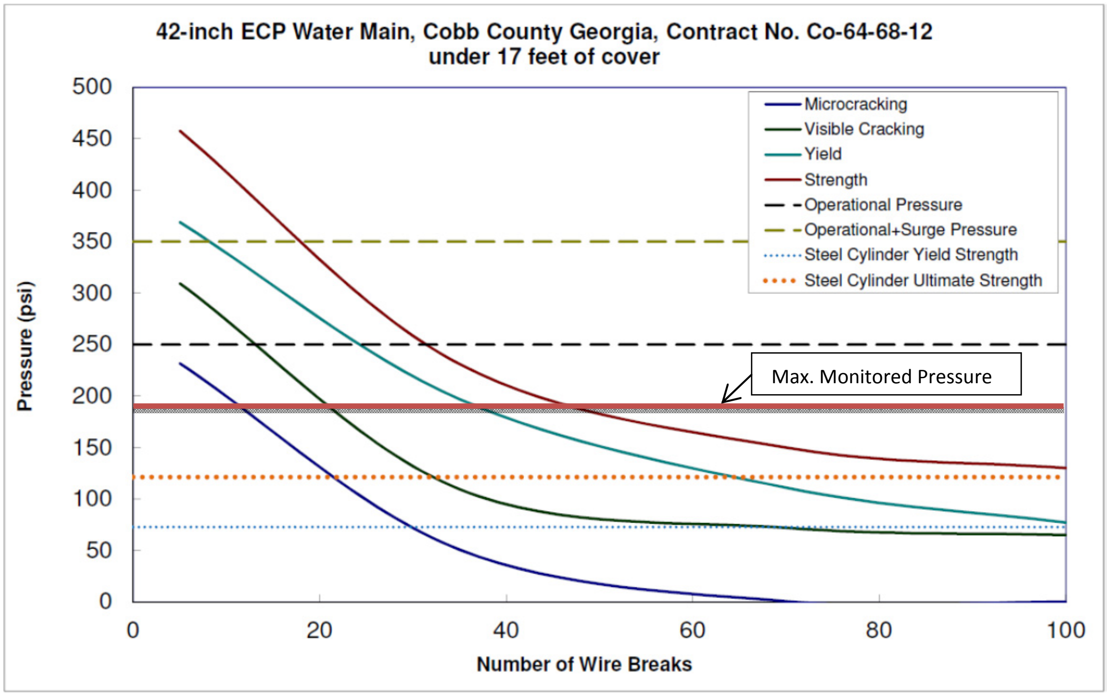

Figure 6. Cobb County, GA 1,067 mm (42-Inch) Transmission Main, Structural Model.

Quantification of the real operating and transient surge pressures aid in the structural evaluation of the transmission main with consideration of any detected damage to help make actionable decisions as to the rehabilitation or the continued service of each individual pipe in the transmission main.

Case Study 4 – WSSC, MD – 1,371.6 mm (54-Inch) PCCP Transmission Main. A transient pressure logger was installed on a 1,371.6 mm (54-inch) water transmission main (TM) in Prince George's County, MD as part of an ongoing PCCP Management Program to detect and quantify potential surge pressures on the pipeline. The transient pressure logger settings are summarized in Table 6.

Table 6. WSSC, MD TM Pressure Logger Settings.

Parameter	Setting
Pressure Sensor Range	-15 to 300 psi
Sample Rate	1,000 Hz (1 mSec interval)
Standard Recording Interval	1 mSec
Transient Trigger ΔP	N/A
Transient Trigger Δt	N/A

Transient pressure monitoring of this PCCP pipeline began in March 2011, along with acoustic monitoring of the PCCP prestressing wires. Beginning on June 30, 2013 acoustic activity on this pipeline began to increase, an increased rate of prestressing wire breaks was recorded by the installed acoustic fiber optic (AFO) monitoring system. Around July 11, 2013 the prestressing wire break rate increased further. This increased rate of wire break activity is shown in Figure 8. The line shows the cumulative acoustic prestressing wire break trend and the bars show the total monthly acoustic wire breaks.

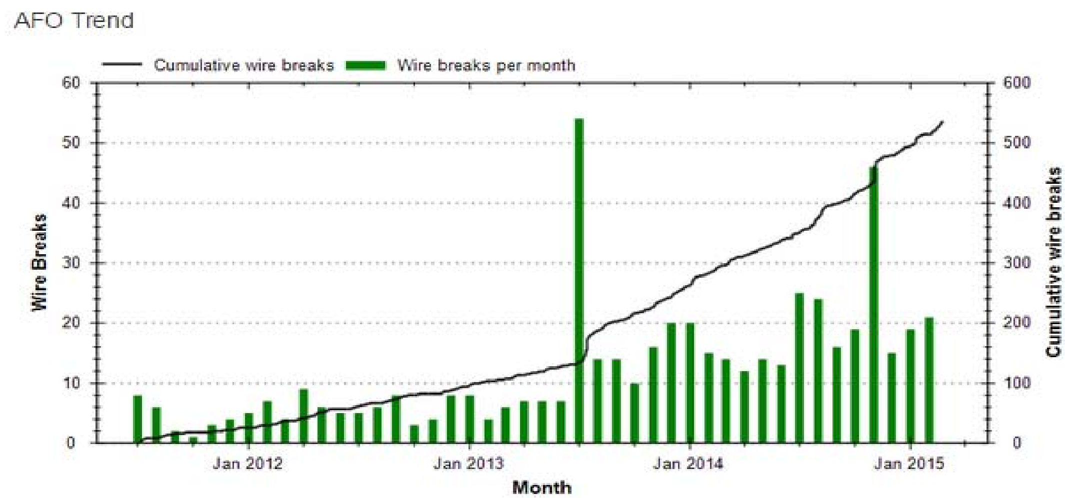

Figure 7. WSSC 54-Inch PCCP Transmission Main AFO Wire Break Activity.

The increase in prestressing wire break activity is statistically significant. Figure 8 shows the transient pressure data recorded from June 14 through July 17, 2013. There are several transient pressure events, the first being a re-pressurization on July 2.

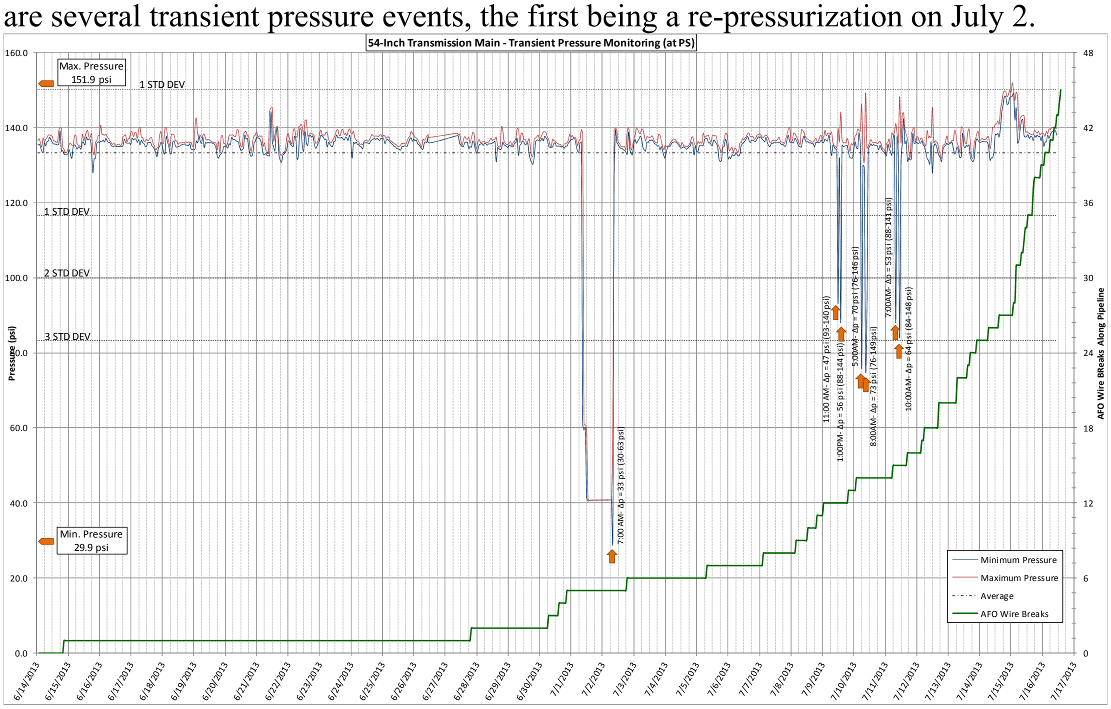

Figure 8. WSSC 54-Inch PCCP Transmission Main, Pressure Data & AFO Wire Break Activity.

Six additional, more severe transient pressures were detected over the period of July 10 through 12. These transients resulted in a lower overall pressure change but in a shorter period of time, with the most extreme being a change of 503 kPa (73 psi) in the period of less than 1 minute. The maximum pressure recorded during these transient events was 1,048 kPa (152 psi). Coordination with the Utility revealed that pump testing / startup operations were being conducted at the nearby pump station in this time frame.

In this case, the increased rate of prestressing wire breaks was concentrated on a single pipe. An emergency shutdown and intervention was conducted and this pipe was replaced. The design operating pressure of the PCCP pipe was 800 kPa (116 psi) and the invert of the pipe was 67.4 meters above mean sea level (MSL). The elevation of the pressure monitor was approximately 36.8 meters above MSL for a static pressure differential of 296 kPa (43 psi). The maximum recorded pressure of 1,048 kPa (152 psi) at the transient pressure monitor translates to 752 kPa (109 psi) at the pipe, 48 kPa (7 psi) below the design operating pressure.

The detected number of prestressing wire breaks on the single pipe where wire break activity was focused was validated after the removal and transport of the salvaged subject pipe section. The correlation of transient pressures with an increased rate in prestressing wire break activity and an increased rate of deterioration is good; however, the pipe was not located at a low point or other feature that would concentrate the effects of a transient pressure event. The pipe was found to be in poor bedding and shallow cover, with a high and possibly fluctuating water table. While pressure is not the root cause of the pipe section's deterioration; the correlation indicates that the significant transient events and peak pressures in conjunction with poor conditions and deterioration ultimately led to this pipe section's replacement.

CONCLUSION

Transient pressure monitoring allows a Utility to monitor the real internal pressure forces imparted on their pipelines. This detection and monitoring is a necessary step in protecting and prolonging the life of these existing assets. Transient pressure monitoring is a useful tool to allow Utilities move to a proactive operating arrangement. The information provided through transient pressure monitoring provides quantifiable support for implementation or improvement of directed maintenance programs for existing surge mitigation systems (i.e. air release valves (ARVs), Surge Tanks, Valves). Correlation of transient pressure data with system operations can also help a Utility make procedural improvements based on the real and current impact on its pipelines. Transient pressure monitoring also provides another quantified input in addition to inspection and assessment results, age, external loading and environment, and original material specifications when evaluating pipeline assets within the framework of condition assessment, rehabilitation, and asset management.

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The Link between Transient Surges and Minimum Pressure Criterion in Water Distribution Systems

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Abstract

The minimum pressure criterion (MPC) is considered as the minimum standard for delivering water pressure when designing water distribution systems (WDSs). This criterion is established by political jurisdictions and is different around the world. A low value of the MPC may reduce water consumption (e.g., faucet, showers, and lawn watering) and also lead to efficient operation through reduced energy use, leakage, and frequency of pipe breaks. However, if this criterion is too low, the system may be more susceptible to low pressure failures, either hydraulic (e.g., an inability to supply the required flow) or safety related (e.g., increasing the risk of an intrusion event and pipe bursts associated with hydraulic transients). Thus, although it may not have been part of the original intent, there is a direct connection between MPC and transients that should not be ignored. This paper looks specifically at the role of MPC and how it affects the system response in transient conditions to raise the awareness about issues that can arise from changes in MPC. First, the definition of MPC and the possible effects of changes in the MPC on WDSs during transient events are briefly explained. Then, two case studies are developed to explore the role of MPC in transient pressures. The results show, not surprisingly, that using surge control strategies is more efficient than increasing the MPC to prevent unwanted surge pressures.

INTRODUCTION

Water distribution systems (WDSs) are designed to provide safe drinking water for domestic consumption. These systems must also provide an adequate supply of water, at an acceptable pressure, to deal with routine and emergency conditions, including fire flow requirements. The standard design approach requires that pressure at any point in the system is maintained within a range whereby the maximum pressure reduces the likelihood of a pipe burst and the minimum pressure provides adequate flow for expected demands.

Indeed, the minimum pressure criterion (MPC) is generally established to ensure for the supply of adequate demand to consumers and possibly, although this is seldom explicit, to prevent of low/or negative pressures during transient events. The MPC is established by political jurisdictions in each country or region and its value changes somewhat around the world. For example, in most provinces in Canada, the MPC is 14 m but in Australia and the UK, the minimum pressure criteria (MPCs) is 20 m and 10 m, respectively (Ghorbanian et al. 2015). Having different MPCs naturally implies that water pressure delivered to customers might be deemed high enough in some countries while the same delivered water pressure in other countries is considered unacceptable. The benefits of reducing the MPC may include decreasing demands, e.g., faucet, showers, and lawn watering, and also improving system performance, i.e. reduction in energy use, leakage, and the frequency of pipe breaks. However, on the negative side, lowering this criterion may cause consumer complaints and make the system more susceptible to low/negative pressure during transient events. Therefore, there is a link between transient pressures and the MPC that cannot be completely ignored.

Indeed, low MPC can put the system at risk during transient events: a risk to the pipeline, to its associated hydraulic devices and to those in their vicinity, and a risk of water contamination and thus to human life. Reduction in the MPC may allow the occurrence of vapor pressure in a transient event, which can lead to column separation in pipeline systems, particularly at specific locations such as closed ends and at high points or knees (changes in pipe slope). In the column separation process, two or more liquid columns are separated by a vapor cavity and then, after wave reflection, the sudden velocity change caused when these liquid columns rejoin, or when one liquid column collides with a closed end, tends to cause an instantaneous rise in pressure (Wylie and Streeter 1983 and Chaudhry 1987). This pressure rise travels as a wave through the entire pipeline and often forms a severe load for individual pipes and supporting structures. Although water column separation and collapse is not common in large networks, this does not eliminate the risk. Another impact of lowering the MPC is to increase in the risk of an intrusion event associated with hydraulic transients. A contaminant may intrude into a WDS through a variety of pathways including submerged air valves, leak points, repair and installations, faulty seals, joints, and service connections when the pressure is low/negative (Thomason and Wang, 2009). A low/negative pressure may be initiated by a pump power failure, a pipe replacement, a valve closure/opening, or demand variations. Gullick et al. (2004) monitored pressure for 43 sites in 8 WDSs and reported 21 negative pressures that lasted less than 3 minutes mainly caused by pump shutdowns. Clearly, not only negative pressures but also water column separation are unwanted in pipeline systems and should be eliminated to the extent practical either by employing surge control strategies or by increasing the steady state pressure. If transient pressures were better controlled using surge control techniques, the system become less vulnerable to the value of MPC; in this context, designers could sometimes reduce the MPC , and still be in a better condition. This paper explores how the MPC affects transient pressures and briefly reviews how destructive transient pressures may be controlled to limit down surge pressures to an acceptable limits even when the MPC is relatively low.

THE ROLE OF MPC IN TRANSIENT PRESSURES

A MPC is generally used in WDSs design to achieve safe, reliable, and economic operation. However, rapid flow changes during transient events generate propagating pressure waves, which have both positive and negative phases as shown in Figure 2. The pressure fluctuations in Figure 2 are produced by a sudden valve closure (i.e., with a closing time of 2 seconds), located at the end downstream of the pipe, occurs in the simple system shown in Figure 1 (the unrealistic negative pressures in Figure 2 is interpreted in the next section). Pressure fluctuations during transient events often violate the regulation of minimum standard for water pressure (Figure 2). To some extent at least, pressure transients in WDSs are inevitable and often most significant at pump stations, control valves/hydrants, and in locations with low static pressures. To minimize a system's susceptibility to surge pressures and to efficiently control down surges to a minimum acceptable level, surge control strategies are often adopted.

Surge control strategies have been divided into three categories: engineering strategies, maintenance strategies, and operational strategies. Engineering and system design strategies include installing surge control devices, using larger diameter pipes, and installing different pipe material. Devices such as surge anticipation valves, pressure relief valves, air release/vacuum valves, surge tanks, and air vessels are often used to control surge pressures in pipeline systems. In maintenance strategies, repair practices are important for the safe and efficient operation of water pipelines systems since deterioration of pipelines is a natural process. Pipelines deterioration often increases the number of pipeline bursts. Therefore, the condition assessment of pipeline interiors, e.g., employing hydraulic transient models for quantifying levels of deterioration (Gong et al. 2013), can be useful for planning rehabilitation or identifying critical points to bursts in water pipeline systems. Operational practices include adjusting the settings of valves, starting and stopping pumps, and operating fire hydrants which are performed as part of the routine operation. A reduced rate of flow change, through slower valve action, proper hydrant operation, and things like using VFDs (variable frequency drives) or increased inertia in

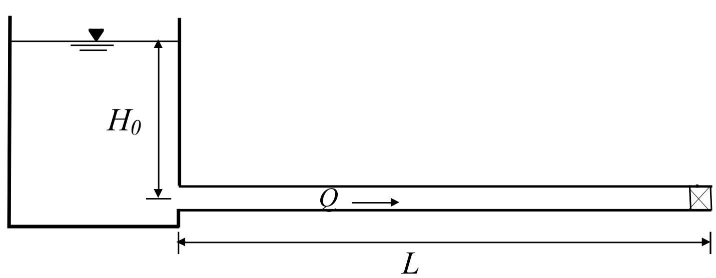

Figure. 1. Simple system configuration (water depth in the reservoir H_0 = 30 m; flow rate Q = 0.5 m^3/s, length L= 1000 m, pipe diameter D = 0.65 m, Darcy-Weisbach friction factor f = 0.015, and wave speed a = 1000 m/s)

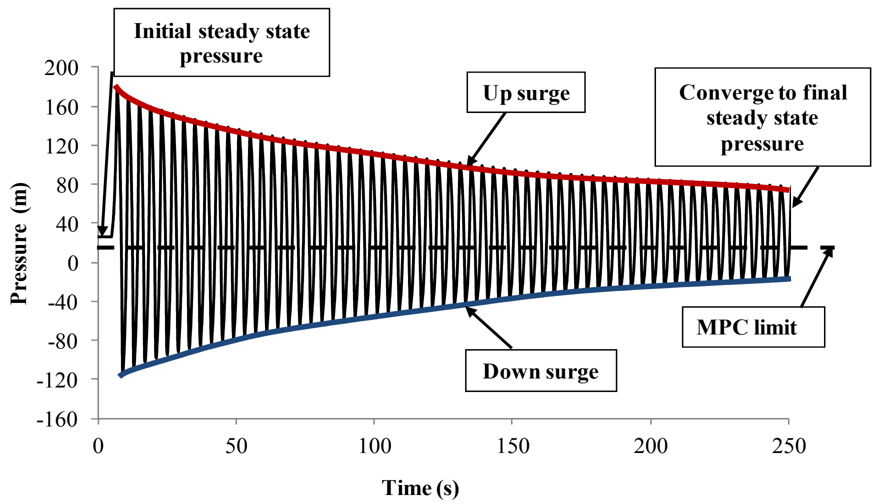

Figure 2. Minimum and maximum transient pressure waves

pumps, are all potentially effective solutions to many problems associated with surge pressures (Wylie and Streeter 1983).

The transient pressures can be controlled by the aforementioned techniques. Minimum transient pressures can be controlled either using surge control strategies or increasing the steady state pressure throughout the system. Two case studies are now presented to explore the role of MPC in transient pressures. To determine the results of transient analysis, a transient model was developed using the method of characteristics (Wylie and Streeter 1983)

Case study 1: series pipeline system. To explore and illustrate how the value of MPC affects the system response during transient events, the series pipes system shown in figure 3 is considered. The length, wave speed, and Darcy–Weisbach friction factor for each pipe are 1000 m, 1000 m/s, and 0.015, respectively. For simplicity, the elevations of all nodes are set to be 0 m. The reservoir water level is at 23.5 in case that the MPC is set to 10 m at the most downstream node. To meet the higher MPCs at node 4, the reservoir level is increased. To introduce transient condition into this case study in a simply way, an almost sudden valve closure (1 s) at the node 4 is initially considered. Figure 4 depicts the pressure envelopes throughout the pipeline system caused by the severe transient condition. The pressure in the pipes becomes unrealistically negative which needs to be either carefully interpreted, or the model improved by including column separation. Fortunately, however, this further complication is often not required, since the main role of the transient analysis is to simply identify whether there is a problem. Clearly figure 4 shows that, no matter what values are plotted, sudden changes in the flow rate can induce powerful and destructive forces into a pipe system.

The primary resistance against up surge pressure is pipe's strength, which is related to its material, wall thickness and general condition. To avoid destructive down surge effects, the valve must be operated slowly, and/or the steady state pressure can be increased

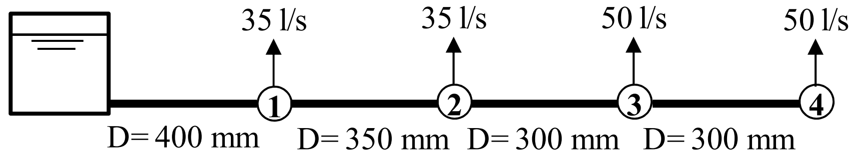

Figure 3. Series pipes system (Gupta and Bhave 1996)

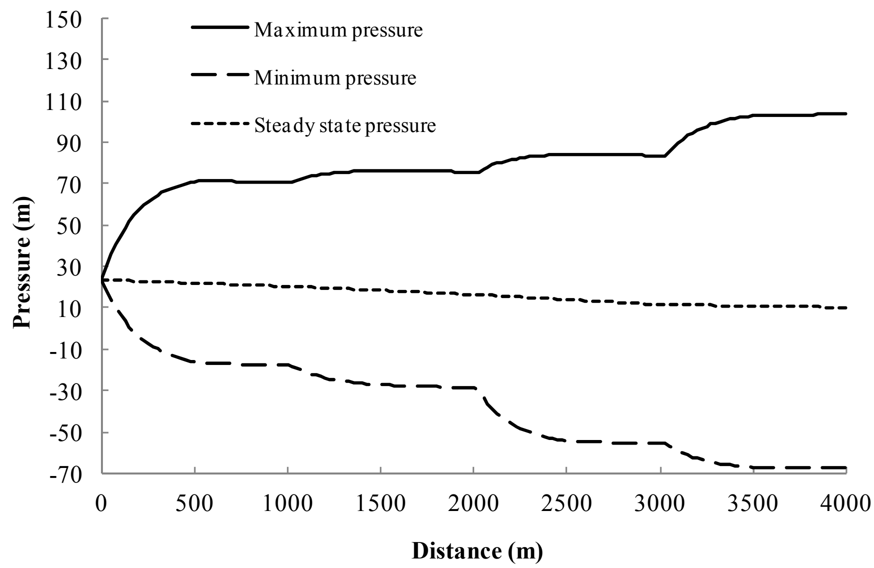

Figure 4. Transient response caused by the downstream valve closure

throughout the system. Figure 5 shows the closure time of the valve, for the series pipes system, against different MPCs in case that the down surge is intended to maintain at 5 m. Not surprisingly, the time of closing valve decreases as the MPC increases. If, in a system, the steady state pressure reduces, the valve's closure time should be increased to make a considerable increase of the down surge pressure, thereby, the minimum transient pressures is maintained at the desired level. Figure 6 shows the case that the steady state pressure increases in order to raise down surge pressures. As illustrated, the down surge pressures still remains negative even the MPC increases as much as 3.5 times. Therefore, increasing the MPC in WDSs design may not be as efficient as adopting minimal surge control strategies to prevent unwanted surge pressures during transient events. Clearly dramatic actions often have consequences even in systems with considerable pressure.

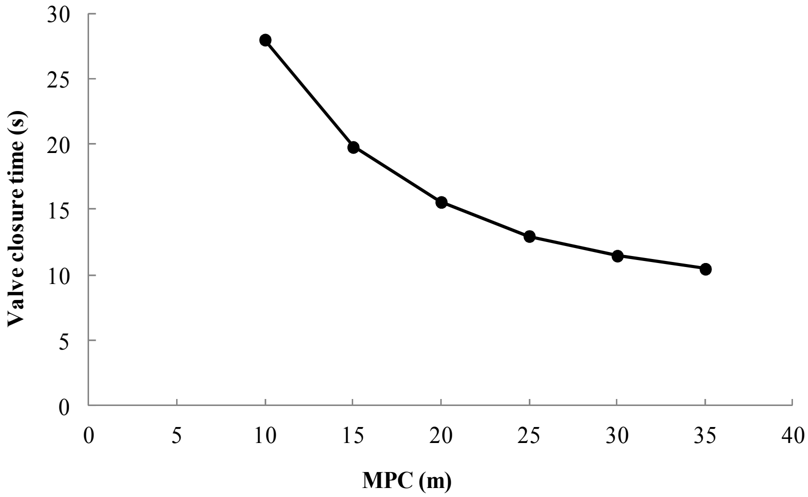

Figure 5. Valve closure time versus different MPCs

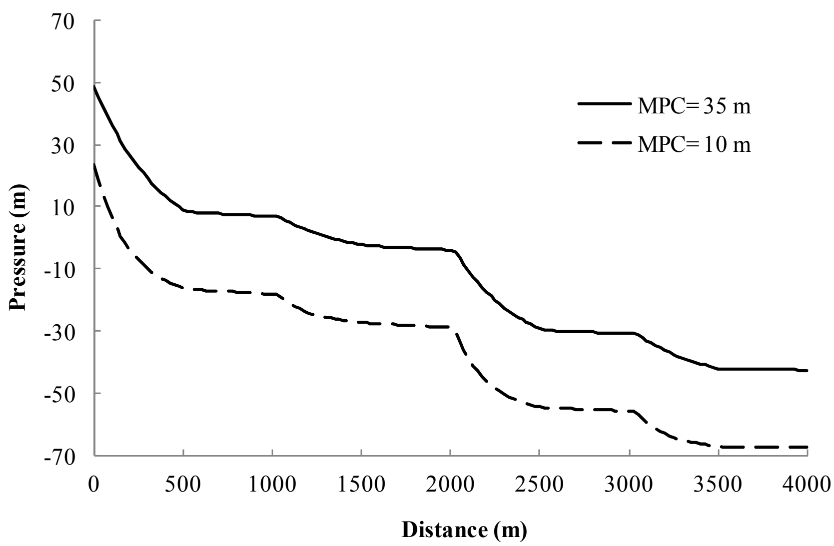

Figure 6. Down surge pressures for different MPCs

Case study 2: the New York City water supply tunnels. The second study network is shown in figure 7; it was first studied by Schaake and Lai (1969) in order to develop their model for optimum design of the primary water distribution system of New York City. The tunnels form is a gravity system that draws water from the Hillview reservoir at node 1.The primary tunnel system consisted of City Tunnels number 1 and number 2, and 19 nodes. The system topology for pipes and demands at each node are set according to Dandy et al. (1996). All junctions are located at the same elevation (0 m). The reservoir head is 48 m to maintain the MPC of 15 m during fire flow events throughout the system. The large demands, pipe flows and large diameters in the test network imply that the network is a skeletonized model and only large trunks are considered. To introduce transient conditions into the system, a set of hydrants operation is considered at nodes 19 and 17 in order to consider the severity of transients due to a couple of fire hydrants operation. It is assumed that the fire flow requirements at nodes 19 and 17 are 1.5 m^3/s and 2.5 m^3/s, respectively, and each hydrant takes 2 s to be open. Opening hydrants in 2s to provide these high values of fire flow is just an assumption and is not realistic; however, in practice it takes more time to reach these flow rates.

Figure 8 shows the transient response in the system at nodes 16, 17, and 19. As expected, there are significant transient effects within the network, i.e. loss of pressure, due to opening of hydrants at nodes 17 and 19. Due to the demands increase at nodes 17 and 19, a reduced pressure wave moves through the system. This wave is reflected from the upstream reservoir and then propagates back and forth in the system. As indicated in figure 8, the pressure dropped at the non fire flow node (node 16) confirming the idea that simultaneous operation of fire hydrants would increase risk of loss of pressure in water networks. As can be seen from the figure, the pressure head falls below 15 m during the transient event although this value is enforced to be the MPC in steady state design of the network.

The issue of operating speed of hydrants to prevent low pressure in the system has been devoted more attention. In this case study to maintain down surge pressures at 10 m, the hydrants at nodes 17 and 19 should be gradually open in 30 s and 110 s, respectively. Figure 9 depicts the transient pressures at nodes 16, 17, and 19 in case that the opening time of hydrants is extended. As shown in the figure, with increasing the opening time of hydrants, the down surge pressure can be maintained at the desired level (e.g., 10 m). It is possible to determine an approximate minimum safe value for the time to operate a valve in order to protect systems against destructive transient pressures (Wylie and Streeter 1983, and Goldberg and Karr 1987). If $t > 2L/a$, where t (s) is the opening time of valve, L (in metres) is the characteristic length of the network, and a is the wave speed for the pipes (in m/s), there can be a considerable reduction of surge pressures in water networks. The characteristic length of the network may be the sum of the pipe lengths from the source of the surge to the upstream reservoir or the energy source of the system. However, determining a specified opening time for every hydrant is a challenging task since there are thousands of network configurations in which the characteristic lengths are different. Although fire crews have been trained on proper hydrant operation, this does not protect the system against low transient pressures due to human errors. To make the system safe during hydrant operations, there should be a device for the control of the down surge pressures at the desired limit even if the fire crews try to open the hydrant as fast as they are able to. This device should be portable to be quickly attached to the hydrant and is able to control minimum transient pressures in different system configurations. This calls for more

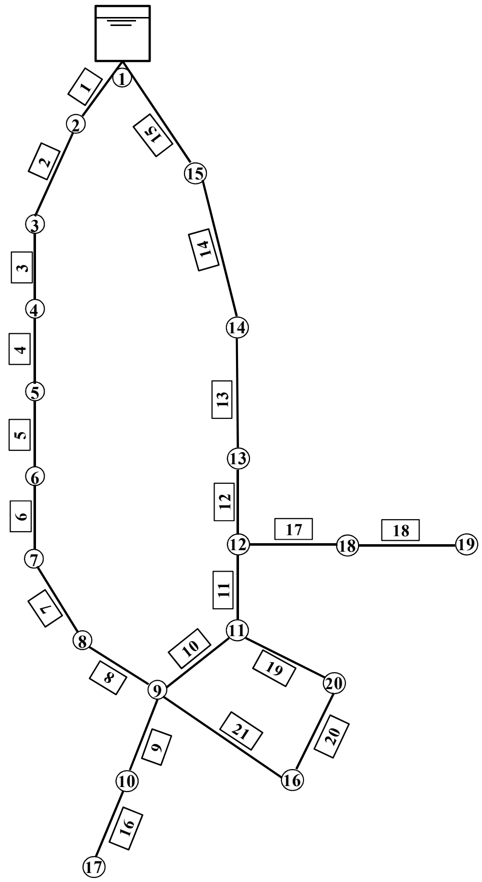

Figure 7. New York City water supply tunnels

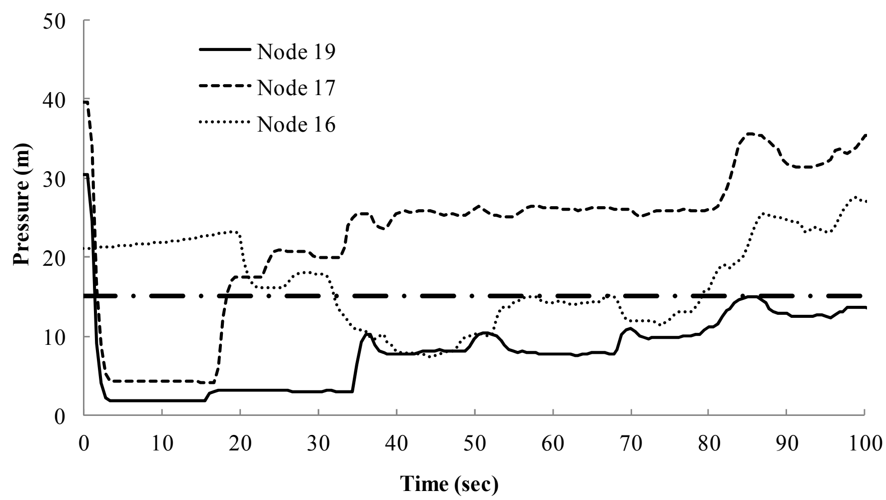

Figure 8. Surge pressure profiles due to hydrants operation

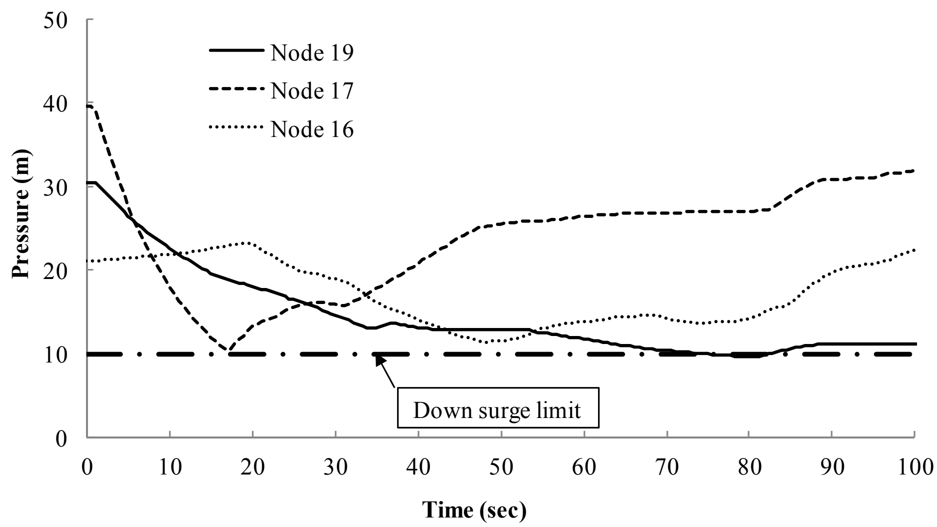

Figure 9. Pressure transient profiles with controlled opening the hydrant

investigation to develop a surge limit control algorithm in a manner that the down surge is controlled in a predetermined level during hydrant operations.

CONCLUSIONS

The role of a MPC is to lead to a reasonable design process and outcome. But as systems have aged, there is a desire to reduce the MPC to save energy costs and reduce the stress on pipeline systems. But lowering the MPC obviously often means systems will have lower pressure, thereby making them more susceptible to negative pressures and contaminant intrusions during transient events. MPC are often violated during transient events due to pressure fluctuations and some care might be needed to define exactly what MPC limits really mean. Consequently, there is an interesting link between transient pressures and the MPC that cannot be completely disregarded. The hydraulic transient response in WDSs is strongly sensitive to system specific characteristics. These destructive transient pressures can be controlled either using surge control strategies, which some of them involve design and operational considerations and some also use the addition of surge protection devices, or sometimes by increasing the steady state pressure throughout the system.

The results clearly show that sudden changes in the flow rate can induce dramatic forces in a pipe system, forces that are quite capable of causing unacceptable operation and even of destroying equipment and components. Transient events can also put water systems at risk of loss of pressure even if systems are normally operated under high pressures. The results indicate that, not surprisingly, increasing the MPC in WDSs design may be inefficient as a surge control strategy. The risk of loss of pressures due to simultaneous operation of fire hydrants can be controlled by extending the opening time of hydrants. However, determining a specified opening time for every hydrant is a challenging task since there are many hydrants scattered at different locations of WDSs including different network configurations. Developing a surge limit control algorithm, to control the down surge pressures during hydrants operation, would seem a worthwhile task. This paper highlights the notion that even those WDSs that are operated under low pressures have risk of high pressure transients, but that transient pressures can be efficiently controlled using surge control strategies.

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Metrics for the Rapid Assessment of Transient Severity in Pipelines

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Abstract

Concepts relating to energy transformations within built and natural systems have been some of the most fruitful in the history of science and engineering. The property of energy summarizes essential changes both in a system's state and key interactions with its environment. Traditional unsteady flow analyses, based on momentum and continuity relations, have been dominated by considerations of wave mechanics, such as unsteady fluid friction which is typically accommodated via adjustments to the momentum equation. The current paper demonstrates how conventional analyses can be supplemented with metrics that can provide a complementary understanding of transient flows. Specifically, this study considers the classical Joukowsky equation, mass oscillations, and the role of energy in analyzing the performance of transient protection devices. The goal is to gain insight by considering energy transformations and interactions.

Keywords: Pipeline systems; Transient analysis; Transient protection; Energy measures; System performance; Simulation; Evaluation metrics.

INTRODUCTION

Transient flow in closed-conduit systems is often too complex to be easily understood due to the interaction of compression waves, boundary conditions, and flow behaviour. One hindrance to a more complete understanding is the fact that experimental and numerical approaches only provide localized perspectives and largely ignore system-wide interactions within a network. Even in simple pipe systems, local transient responses may be the result of the interaction of waves with different origins from across the system (e.g., boundary conditions, friction, compressibility effects). Factors contributing to the behavior at a point of interest are difficult to understand unless the local response is decomposed into a series of waves representing each contributing factor. While beneficial, such an approach is often awkward; a more holistic view is considered here.

In deriving the integrated energy equation for unsteady-compressible pressurized flow, Karney (1990) proposed studying a network's energy fluxes to better understand its transient hydraulics. This involves balancing the net energy flux entering the system with the time rate of change its kinetic energy, internal (i.e., elastic) energy, and dissipative forces. It was shown that an insightful understanding of both a system's transient response and the influence of specific factors can be gained in this way. This concept can be extended to re-evaluate existing derivations, analyze mass oscillations, and study how transient protection devices act as energy sources and sinks to alleviate transient pressures. The present study investigates the utility of supplementing conventional transient analyses with consideration of energy interactions. Although sophisticated commercial software allow the transient hydraulics of complex pipe networks to be simulated with ease, such analyses alone are incomplete without analytical insights.

ENERGY EQUATION FOR UNSTEADY-COMPRESSIBLE FLOW

By manipulating the governing equations of one-dimensional unsteady-compressible flow in closed conduits, Karney (1990) derived the following energy equation:

$$\frac{dU}{dt}+\frac{dT}{dt}+D'+W'=0 \quad [1]$$

where *U* is the internal elastic energy (J), T is the kinetic energy (J), *D'* is the rate of energy dissipation (J/s), and *W'* is the rate at which work is done to force the fluid through the conduit (J/s). The terms in Equation [1] are respectively given by

$$\frac{dU}{dt}=\frac{\rho A}{2}\left(\frac{g}{a}\right)^2\frac{d}{dt}\int H^2dx \quad [2]$$

$$\frac{dT}{dt}=\frac{\rho A}{2}\frac{d}{dt}\int V^2dx \quad [3]$$

$$D'=\frac{f\rho A}{2D}\int|V|^3dx \quad [4]$$

$$W'=\rho gAV(L,t)H(L,t)-\rho gAV(0,t)H(0,t) \quad [5]$$

where *H* is the piezometric head (m), *V* is the average flow velocity (m/s), *a* is the acoustic wave velocity (m/s), *A* is the conduit's cross-sectional area (m^2), *D* its diameter (m), *L* its length (m), ρ is the fluid's density (kg/m^3), *g* is the acceleration due to gravity (m/s^2), *f* is the Darcy-Weisbach friction factor, *x* is distance along the conduit (m), and *t* is time (s). The spatial integration bounds are $x = [0, L]$.

EXAMPLE APPLICATIONS USING AN ENERGY APPROACH

Unlike considering momentum and continuity as is done within typical water hammer models, an energy-based approach is advantageous in that it provides a different perspective that simultaneously combines both of the aforementioned characteristics within a single measure. In deriving the energy relations for transient closed-conduit flow, Karney (1990) showed that an analysis of a pipe network's energy fluxes during valve closure events leads to a different interpretation and therefore understanding of the underlying hydraulics. Similar concepts are illustrated here for four examples.

Alternative Derivation of the Joukowsky Equation

The classical Joukowsky equation, which is derived by applying Newton's second law to a control volume moving at a conduit's acoustic wave speed (Wylie & Streeter 1993), is given by

$$\Delta H=\pm\frac{a}{g}\Delta V \quad [6]$$

where ΔH is the instantaneous change in head (m) due to a sudden change in velocity ΔV (m/s) that occurs within a time period less than the conduit's characteristic time $T = 2L/a$ (s). Interesting, the relation also arises naturally from energy considerations.

Consider the simple system shown in Figure 1. The system, which contains water, is initially at steady state with velocity V_0 and the downstream valve fully open.

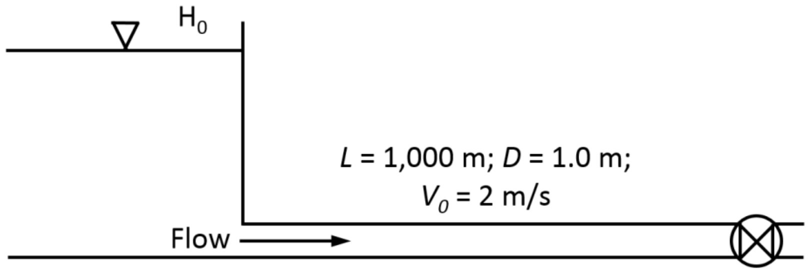


Figure 1: Simple Reservoir-Pipe-Valve System

Sudden valve closure induces transient conditions whereby a positive pressure wave propagates away from the valve at the acoustic wave velocity. The sudden head rise accompanying this operation can be determined using Equation [6]. When the wave reaches the upstream reservoir, the fluid column comes to rest (although only for a brief instant) and the fluid's total kinetic energy is approximately zero. This gives rise to the question of where all of the system's initial energy is. The Joukowsky equation alone is unable to answer this. In using an energy approach, it is found that the system's initial kinetic energy becomes internal energy contained within both the fluid as elastic potential energy and the conduit's walls as strain energy.

In order to formulate the problem such that the derivation considers energy interactions, the system's initial kinetic energy can be balanced with the sum of the energy components stored in the water column and conduit:

$$E_0 = \frac{\pi}{2}\rho L R^2 \Delta V^2 \quad [7]$$

$$E_1 = U_f + U_p \quad [8]$$

where: E_0 and E_1 are the total energies (J) at times $t_0 = 0$ s and $t_1 = L/a$ (s), respectively; R is the conduit's radius (m), and; U_f and U_p are the internal energies of the fluid and conduit wall (J), respectively. The latter two terms are given by

$$U_w = L\frac{\pi R^2}{2K}\Delta P^2 \quad [9]$$

$$U_p = L\frac{\pi R^3}{eE}\Delta P^2 \quad [10]$$

where P is the pressure (Pa), e is the conduit's wall thickness (m), K is the fluid's bulk modulus (Pa), and E is the elastic modulus of the conduit (Pa). By combining Equations [7] through [10] and manipulating the resulting expression, the following is obtained:

$$\Delta P = \frac{\rho\sqrt{\frac{k}{\rho}}}{\sqrt{1+\frac{K}{E}(D_R - 2)}} \times \Delta V \quad [11]$$

where $D_R = e/D$ is the conduit's dimension ratio. The term preceding the velocity term is the conduit's acoustic wave velocity. In addition to being relatively simple, this derivation also relates how the fluid's kinetic energy transforms into the fluid's elastic potential energy and the conduit's strain energy.

For the system in Figure 1, given that the bulk fluid modulus for water is $K = 2 \times 10^9$ Pa, the wave velocity and energy components for different conduit materials and dimension ratios can be evaluated and compared. For example, consider PVC and steel which have elastic moduli of $E_{PVC} = 2.7 \times 10^{-9}$ Pa and $E_{steel} = 2 \times 10^{11}$ Pa, respectively. Figures 2 and 3 illustrate the wave speeds and energy components for different dimension ratios for both materials.

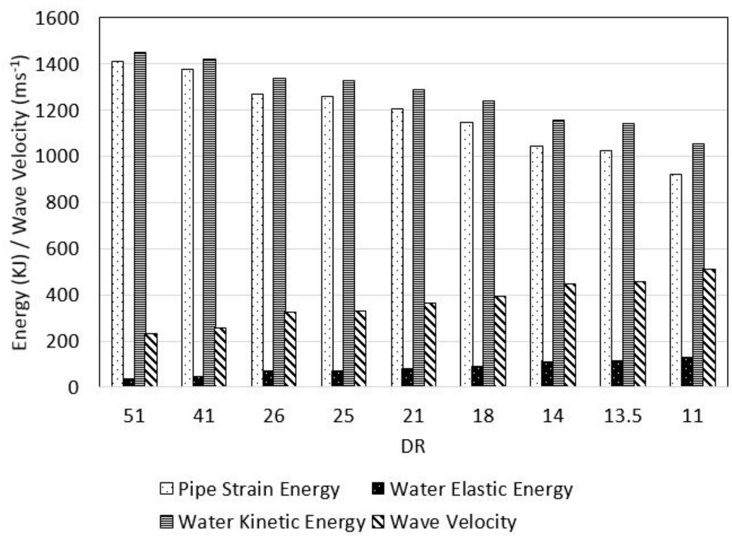


Figure 2: Energy Components and Wave Velocities for PVC Pipe

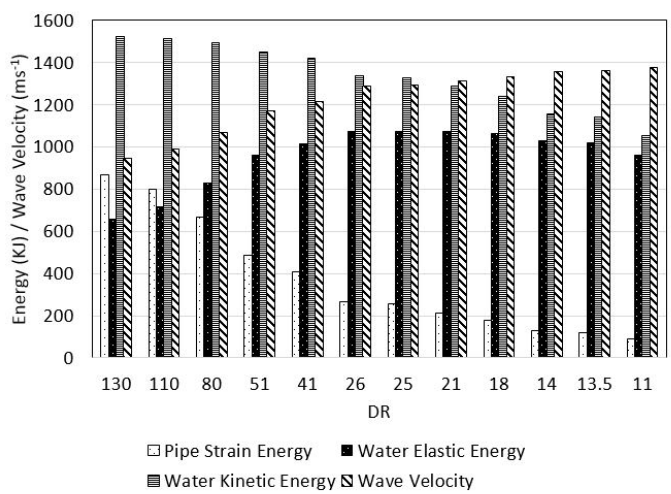


Figure 3: Energy Components and Wave Velocities for Steel Pipe

Figures 2 and 3 show that PVC absorbs more of the fluid's initial kinetic energy as strain energy than steel, while much of the fluid's initial kinetic energy is converted to elastic potential energy in the fluid itself for steel. These differences are due to steel having a much greater elastic modulus than PVC. Thus, PVC absorbs more energy and reduces the magnitude of transient pressures, makes it favourable for sudden changes.

Mass Oscillations

In cases where flow changes occur gradually over time and when there are large storage volumes, compressibility effects are negligible and inertial effects being the predominant dynamic characteristic. This is especially the case for mass oscillations in storm water conveyance systems. Such systems comprise conduits and shafts with large storage volumes: during rainfall events, upon becoming pressurized much of the flows' kinetic energy will cause water levels to rise and oscillate in the shafts. The energy equation is also useful for predicting the maximum water level rise in a downstream shaft.

Even a simple example provides physical insight. Figure 4 illustrates a system comprising an upstream water reservoir and an initially empty downstream shaft connected by an empty pipe of length *L* and diameter *D* with a control valve. Upon opening the valve, the tank is rapidly filled. Due to inertia, the tank's water level may actually become greater than that of the reservoir and a mass oscillation process begins. Of interest here is the maximum fluid level in the shaft.

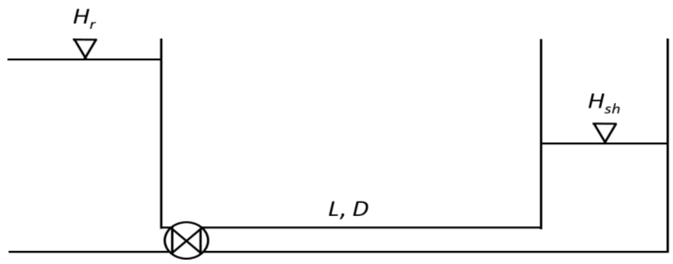


Figure 4: Schematic of a Rapidly Filling Downstream Shaft

In formulating a model for this example, Equation [1] can be simplified and extended to account for the potential energy accumulated in the downstream shaft:

$$\frac{dT}{dt} + \frac{dZ}{dt} + W' = 0 \qquad [12]$$

where *Z* is the potential energy accumulated in the downstream shaft. This formulation ignores the effects of friction which, while not theoretically correct, allows the problem to be simplified such that meaningful insights can be gained. The components of Equation [12] are given by

$$\frac{dT}{dt} = -\frac{\rho}{2\Delta t} L_T A_T V^2 \qquad [13]$$

$$\frac{dZ}{dt} = \frac{1}{2\Delta t} \rho g A_{sh} {H_{sh}}^2 \qquad [14]$$

$$W' = -\frac{1}{\Delta t} \rho g H_{sh} A_{sh} H_r \qquad [15]$$

where *L* is the pipe's length (m); *A* is the pipe's area (m); A_{sh} is the shaft's area (m^2); H_{sh} is the height of water in the downstream shaft (m), and; H_r is the height of water in the upstream reservoir (m). Substituting Equations [13] through [15] into Equation [12] and solving the resulting quadratic expression yields

$$H_{sh} = H_r + \sqrt{{H_r}^2 + \frac{LAV^2}{gA_{sh}}} \qquad [16]$$

Note that the term *V* in Equation [16] represents the velocity at the instant the tunnel becomes completely full and the downstream shaft starts being filled. This simple analytical formula estimates the maximum water level in the shaft. The first term, the upstream reservoir's head, is an energy source, while the second term represents the difference between the maximum shaft water level and the reservoir's water level (the "overshoot"). The latter term provides some insight into the parameters that affect the magnitude of the overshoot. For example, Equation [16] suggests that increasing the upstream reservoir level has an almost linear influence, while changing either the shaft or pipe area will directly alter the maximum water level.

In addition to the analytical approach, a numerical exploration was undertaken using the model proposed by Malekpour and Karney (2011). This model solves the governing momentum and continuity equations using the method of characteristics (MOC). Because the length of the water column grows with time, a dynamic computational mesh that actively adapts to the water column's length was employed. To evaluate how well Equation [16] predicts the maximum head rise, simulations were performed for various pipe lengths, shaft diameters, and reservoir heads while assuming a constant friction factor of $f = 0.018$. Table 1 provides a summary of the cases analyzed, while Figure 5 presents a comparison of the analytical and simulation results.

Table 1: Summary of Cases Analyzed

Case No.	Tunnel Dimensions		Reservoir Head (m)	Shaft Diameters (m)
	Diameter (m)	Length (m)		
1	5	1000	5.0	2, 5, 10, 40
2	5	2000	9.5	2, 5, 10, 40

For both cases in Table 1, the reservoir head is intentionally selected such that when the tunnel becomes completely full the velocity established in the system is 5 m/s. Figure 5 shows that the analytical approach generally underestimates the maximum head rise, but that it compares well with simulation results. Such an analytical formula can be useful for estimating the overshoot prior to implementing a detailed numerical model.

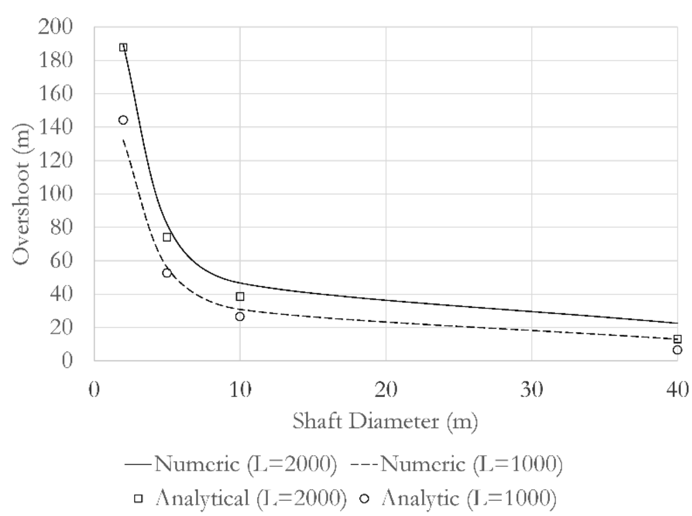
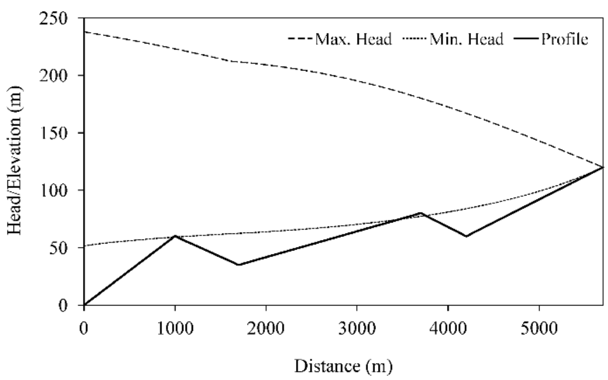


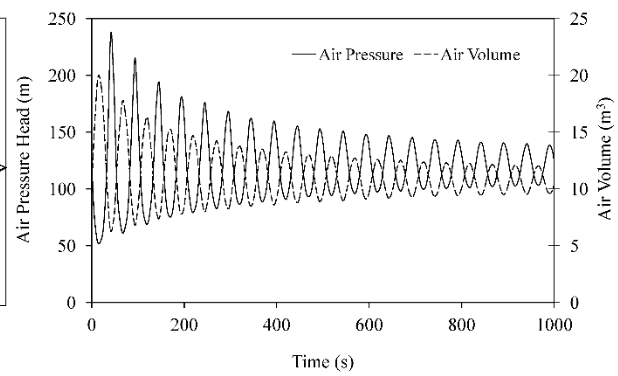
Figure 5: Comparison of Analytical and Numerical Results

Pumping Pipeline with an Air Chamber

From an energy perspective, the pump in a pumping pipeline is an energy source that supplies energy to the fluid. Upon removal of the energy source (e.g., pump failure due to a power outage), there is a sudden stoppage of flow at the energy source that is not immediately communicated throughout the rest of the pipeline: that is, at the instant when the energy source is removed, the fluid continues to discharge at the downstream end of the pipeline. This is possible because the pipeline itself supplies the energy for continued discharge; however, this comes at the cost of a significant decrease in head that travels along the pipeline in the form of a compression wave. The magnitude of this downsurge can be alleviated or even mitigated by providing a temporary energy source, such as an air chamber, flywheel, or surge tank, that relieves the pipeline's energy contribution to the fluid. To demonstrate this concept, this and the following section investigate supplementing conventional transient analyses of pumping pipelines with an energy approach to evaluate the performance of different protective devices.

The first pumping pipeline investigated, which is from Karney et al. (2014), is shown in Figure 6a. This system comprises a pump station with three parallel pumps, an air chamber on the pump station's discharge side, and a pipeline with an undulating profile that connects two water reservoirs with heads of 0 m and 120 m, respectively. The pipeline connecting the two reservoirs has a length of L = 5,700 m, a diameter of D = 1.0 m, a Darcy-Weisbach friction factor of f = 0.018, and a wave speed of a = 1,000 m/s. The three pumps have a total combined flow of Q_0 = 0.39 m3/s and are each characterized by a rotational inertia of I_p = 3.0 kg-m^2, a speed of ω_0 = 900 rpm, and a rated head of H_0 = 131.4 m.





(a) Maximum and Minimum Head Envelopes (b) Air Chamber Pressure and Volume

Figure 6: Simulation Results for a Pumping Pipeline with an Air Chamber (Karney et al., 2014)

Of interest is a power failure event that induces transient conditions in the system due to sudden pump stoppage. Transient protection devices such as pressurized air chambers temporarily supply energy to a system when the system's primary energy source is removed, thus alleviating transient pressures. However, understanding the behaviour of such devices is not always straightforward, especially for complex pipe networks. To aid interpretation, an energy approach is used to analyze simulation results and provide insight into how an air chamber helps control negative pressures by augmenting the system's hydraulics.

A water hammer model was used to simulate the transient hydraulics of the pumping pipeline. The air chamber was modeled with an initial air volume of V_0 = 10 m^3 using the polytropic law with a polytropic exponent of γ = 1.2. This initial volume was selected via a trial-and-error approach so as to mitigate negative pressures. The simulation results are provided in Figure 6. Figure 6a shows that negative pressures are entirely mitigated, with the small exception of pressures at approximately 3,800 m from the pump station. The air chamber's head trace shown in Figure 6b is also representative of the head at the pump station's discharge side.

At the first instance when the water column is entirely at rest at a time of 17.5 s, the air chamber reaches its maximum volume of 20 m^3 and minimum head of 51.6 m. From the start of the simulation to this time, numerical results show that 11.9 m^3 of fluid have discharged from the pipeline. In order to determine the amount of energy leaving the system during this period, a reference datum must be defined: for this we will use the minimum head at the pump station's discharge (i.e., H_{ref} = 51.6 m). From here, the energy leaving the system during the first 17.5 s is

$$E_l = mg(H_2 - H_{ref}) = (11.9\ \mathrm{m}^3)(1{,}000\ \mathrm{kg/m}^3)(9.81\ \mathrm{m/s}^2)(120\ \mathrm{m} - 51.6\ \mathrm{m}) \cong 8\ MJ$$

Now, consider if this energy were supplied to the fluid entirely by the system: if that were the case, there would almost certainly be negative pressures. Instead, the air chamber supplies this energy as demonstrated below. Consider the cylinder-piston-air system in Figure 7.

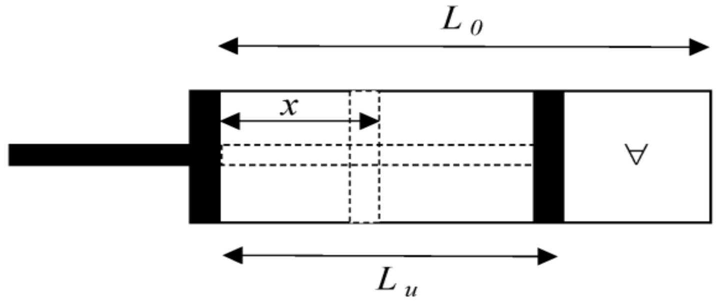


Figure 7: Cylinder-Piston-Air System Illustrating Air Compression (Karney et al., 2014)

The energy stored within the compressed air pocket of volume $\forall$ is given by

$$E_{air} = \int_0^{L_0} A_{ac} P_x dx \qquad [19]$$

where E_{air} is the energy stored in the air pocket (J), P_x is the air pressure (Pa) when the piston is located at a distance of x (m) from its original position, and A_{ac} is the cylinder's cross-sectional area (m^2). The term P_x can be calculated using the polytropic law as

$$P_x = P_{atm}\left(\frac{L_0}{L_0 - x}\right)^{\gamma} - P_{atm} \qquad [20]$$

where P_{atm} is the atmospheric pressure (Pa) and γ is the polytropic exponent. Recall that the air chamber was modeled using γ = 1.2.

By combining Equations [19] and [20] and integrating over the length of the compressed air pocket, Malekpour and Karney (2014) derived the following expression for the energy stored in the air pocket as a function of the air pocket's volume and the atmospheric pressure:

$$E_{air}(\forall, P_{air}) = \frac{\forall P_{atm}}{1-\gamma}\left[\gamma\left(\frac{P_{air}}{P_{atm}} + 1\right)^{\frac{1}{\gamma}} - \left(\frac{P_{air}}{P_{atm}} + 1\right) + 1 - \gamma\right] \qquad [21]$$

Alternatively, Equation [3] can be expressed in terms of piezometric head as

$$E_{air}(\forall, H_{air}) = \frac{\rho g \forall H_{atm}}{1-\gamma}\left[\gamma\left(\frac{H_{air}}{H_{atm}} + 1\right)^{\frac{1}{\gamma}} - \left(\frac{H_{air}}{H_{atm}} + 1\right) + 1 - \gamma\right] \qquad [22]$$

where H_{atm} and H_{air} are the atmospheric and compressed air heads (m), respectively.

The amount of energy supplied by the air chamber is equal to the change in its energy from time t_0 = 0 s to time t_1 = 17.5 s. Using Equation [4], this can be calculated as

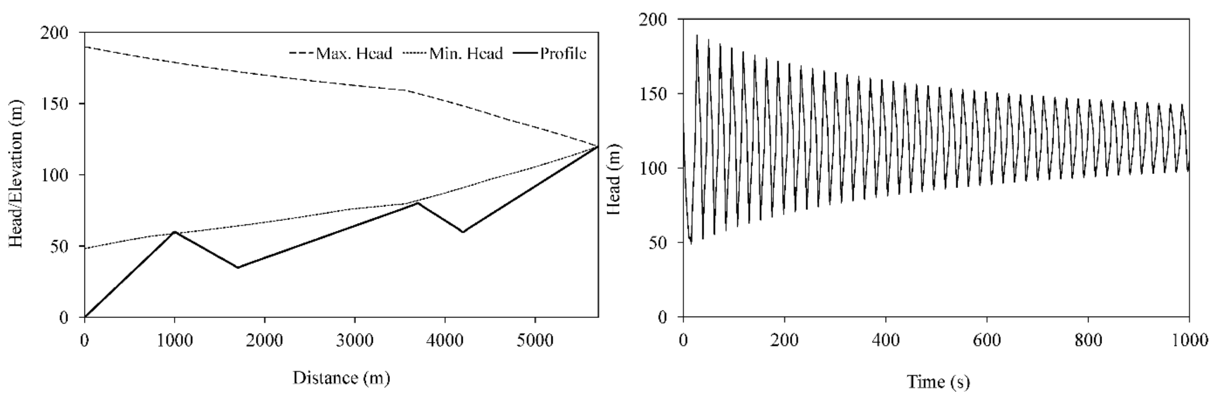
$$\begin{aligned}\Delta E &= E_{air}\left(\forall_{t_1}, H_{air_{t_1}}\right) - E_{air}\left(\forall_{t_0}, H_{air_{t_0}}\right)\\ &= E_{air}(20\ \mathrm{m}^3, 52\ \mathrm{m}) - E_{air}(10\ \mathrm{m}^3, 131\ m)\\ &= -12.1\ \mathrm{MJ}\end{aligned}$$

The difference in energy (i.e., 12.1 MJ - 8 MJ) of 4.1 MJ can be attributed to dissipative forces within the pipeline system (i.e., head losses). The value above is negative because the air chamber itself supplied energy and therefore lost energy to the pipeline system. Additionally, only the first wave cycle was analyzed since this governs the system's head envelopes.

Pumping Pipeline with a Flywheel

In addition to air chambers, other transient protection devices can be used to temporarily supply energy to the fluid when an energy source is removed. In this section, we investigate adding a flywheel to the three pumps in the pumping pipeline system. Adding a flywheel to a pump increases the pump's rotational inertia, and therefore its stored energy, which extends the pumps' run down time thereby alleviating the resulting transient pressures. By increasing a pump's rotational inertia, the pump's ramp-up time during its start-up is also increased: this is undesirable from an operational perspective, yet beneficial from a transient perspective.

This example, like the previous one, is referenced from Karney et al. (2014). Similar to sizing the air chamber in section 3.3, a trial-and-error approach was adopted when selecting a flywheel size that mitigates negative transient pressures. This lead to selecting three flywheels with a total rotational inertia of I_{fw} = 997 kg-m^2. Simulation results from the MOC-based water hammer model are provided in Figure 8.



(a) Maximum and Minimum Head Envelopes (b) Head Time History at the Pump Station

Figure 8: Simulation Results for a Pumping Pipeline with Flywheels (Karney et al., 2014)

In comparing the simulation results in Figures 6 and 8, it can be seen that the minimum head envelopes are relatively similar. Figures 6a and 8a also show that the maximum head envelope for the system with flywheels is less than that for the system with an air chamber, which suggests that the former is more effective as a protective measure.

Similar to the air chamber, the system's performance with flywheels can be studied using an energy approach. The energy contained in the flywheels is given by

$$E_{fw} = \frac{3}{2} I_{fw} \left(\frac{2\pi\omega}{60}\right)^2 = \frac{\pi^2}{600} I\omega^2 \qquad [23]$$

Using Equation [23], the energy supplied to the system by the flywheels from full speed at time t_0 to zero speed at time t_1 can be calculated as

$$\Delta E = E_{fw_{t_1}} - E_{fw_{t_0}} = \frac{\pi^2}{600}(997\ \mathrm{kg \cdot m^2})[(0\ \mathrm{rpm})^2 - (900\ \mathrm{rpm})^2] = -13.3\ \mathrm{MJ}$$

Once again, the energy provided to the pipeline system is negative because the flywheels supplied and therefore lost energy. In comparing the energy supplied by the air chamber (9 12.1 MJ) with that of the flywheels (13.3 MJ), it can be seen that the latter contributed 50% more energy than the former. This variation can be attributed to the pumps' efficiency: the quotient of the energies (i.e., 12.1 MJ / 13.3 MJ) is approximately 90%, which is representative of the pumps' average efficiency during the first transient cycle.

CONCLUSIONS

As a fundamental characteristic of the physical world, energy transformations and interactions provide a unique perspective of the underlying characteristics of many phenomena. In the case of transient flow in pressurized pipe networks, conventional transient analyses can be supplemented by analyzing their energy characteristics. This article investigated four particular cases where consideration for such energy fluxes led to an improved understanding of unsteady-compressible flow behaviour. The first of which involved re-deriving the classical Joukowsky equation for the instantaneous head change due to sudden flow stoppage, while the latter three supplemented transient simulation results with simple energy relations to gain a better understanding of how protective devices alleviate transient pressures.

Despite the insights provided by the present examples, such energy-based approaches can be extended further and applied to both complex pipe networks (e.g., water distribution systems), complex transient phenomena (e.g., unsteady friction, waver interaction, cavitation), as well as systems that experience mixed free surface and pressurized flow (e.g., storm water transmission systems) to gain an improved understanding their behaviour. Additionally, energy transformations can be used to explore transitions between the different transient flow regimes and establish their boundaries and develop analytical formulae for designing protective measures.

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Case Study: Hydraulic Modeling and Field Verification on the Rietspruit-Davel-Kriel Bulk Water Supply Pipeline

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Abstract

Extending the safe service life of aging pipeline infrastructure requires an understanding of the actual structural integrity of the asset, as well as the loading imposed on the pipeline through, amongst other factors, the way in which the pipeline or system is operated. A change in the steady state operating condition of a fluid system, by means of valve or pump operational change, or due to system failure, is communicated to the system by pressure waves, propagating from the point of the origin where the change in steady flow condition was imposed. The system attains a new state of equilibrium, after some time, if the change has not reached destructive proportions. The terms “surge”, “water hammer” and “transient flows” are used synonymously to describe an unsteady flow of fluids in a pipe system. Various transient modeling software packages based on proven mathematical and numerical solutions are commercially available today. Confidence in modeled results can however be improved by comparing actual field measurements with modeled results. This paper describes the hydraulic modeling, field verification and comparison of modeled and measured results achieved on the Rietspruit-Davel-Kriel bulk water supply pipeline for the Department of Water and Sanitation in South Africa. The hydraulic assessment was performed as part of a comprehensive risk based condition assessment project.

PROJECT BACKGROUND AND INTRODUCTION

The Usutu Water Scheme in the Mpumalanga Province of South Africa supplies raw water to various coal fired power stations and towns. The DN1300 (51 in.), pre-stressed concrete non cylinder pipe (PCP) between the Rietspruit and Davel Reservoirs (36.5km) and between the Davel and Kriel Reservoirs (54.4km) was completed in the late 1970's. The Rietspruit-Davel-Kriel (RDK) Transmission Mains form a strategic link in the Usutu Water Scheme (Figure 1). The scheme is owned and operated by the Department of Water and Sanitation (DWS).

Figure 1: Alignment of the Rietspruit-Davel and Davel-Kriel Pipelines

Both transmission mains have experienced multiple failures in the past that were caused by a range of mechanisms.

The criticality and failure history prompted an investigation into the reliability of the RDK Transmission Mains. This was achieved through a comprehensive risk-based condition assessment of the two mains. The assessment employed least disruptive in-line inspection technologies (leak and gas pocket detection and an electromagnetic inspection), various external surveys and advanced engineering assessment techniques, as described in Paper No 249 '…and the kitchen sink. Using a full toolbox to assess a critical bulk water asset in South Africa'. The engineering assessment aimed not only to identify the pipes in need of remediation, but also to infer some of the root causes of distress in order to slow future deterioration and prolong the remaining useful life of the assets. As one of the potential contributing factors, the transient behavior of the pipelines was confirmed through on-site monitoring based upon which a calibrated hydraulic model of the system was compiled. The field verification and hydraulic modelling of the Rietspruit-Davel-Kriel Transmission Main is the topic of this paper.

PIPELINE DESCRIPTION

The Rietspruit-Davel and Davel-Kriel pipelines are both downstream controlled, gravity systems between fixed head nodes (reservoirs). Both pipelines are operated in a similar fashion. Due to the similarity of the two systems, only the field verification and modelling of the Davel-Kriel pipeline is described in this paper.

The pipeline starts at the Davel Reservoir complex and discharges into the Kriel Power Station raw water storage reservoir under gravity. Figure 2 shows the layout of the pipeline as well as the location of key points along the route where field measurements were taken.

Figure 2: Davel-Kriel layout and field verification sites

The longitudinal profile of the pipeline, also illustrating the locations where field measurements were taken is illustrated by Figure 3. The location of pipeline components and the pressure rating of the pipeline based on information obtained from the as-built data are also displayed.

The Davel-Kriel pipeline is downstream controlled by means of a DN1200 actuated Ring Needle Valve (RNV) located at the inlet to the Kriel Reservoir (Figure 4). The control valve is operated remotely in 25% increments with local override capability. The actuator and gearbox arrangement is such that full actuation takes approximately 20 minutes (1200s) to execute.

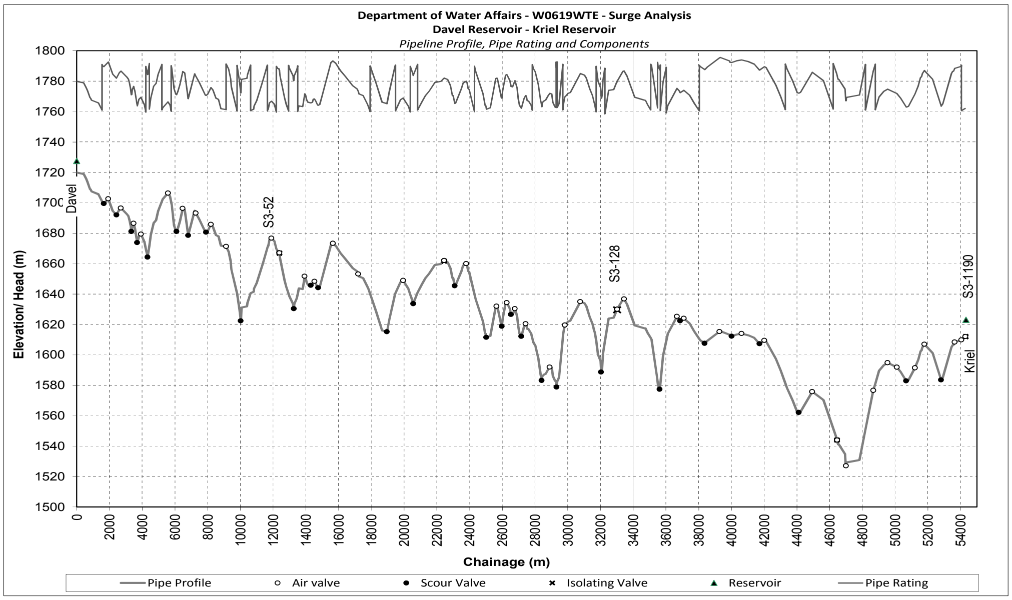

Figure 3: Davel-Kriel, pipe profile, pipe rating, component location and measuring sites

Figure 4: Kriel Reservoir inlet control valve

FIELD VERIFICATION

In order to calibrate the steady state and dynamic models, actual pressures were measured by high frequency pressure loggers, while the flow rates were logged at existing flow meters at the upstream and downstream ends of the pipeline.

The Pipetech TP-1 transient pressure monitors record the variation of pressures within a pipeline and have the ability to 'sense' the approach of a pressure transient and automatically increase its rate of data capturing to ensure that the surge event is accurately recorded as illustrated by Figure 5. The device can therefore be used to measure both static and dynamic pressure variations over long periods without generating extensive data sets.

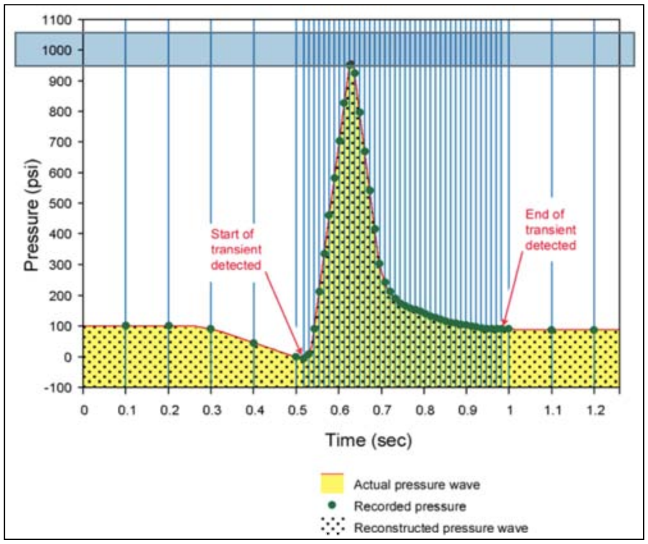

Figure 5: Dual frequency pressure plot

A typical pressure logging site arrangement on the Davel-Kriel pipeline is illustrated by Figure 6.

Figure 6: Typical pressure logging site arrangement

The logging frequency is an important consideration when trying to detect transient events in a pipeline. As illustrated by Figure 7, low frequency pressure logging alone could result in high transient pressures going undetected.

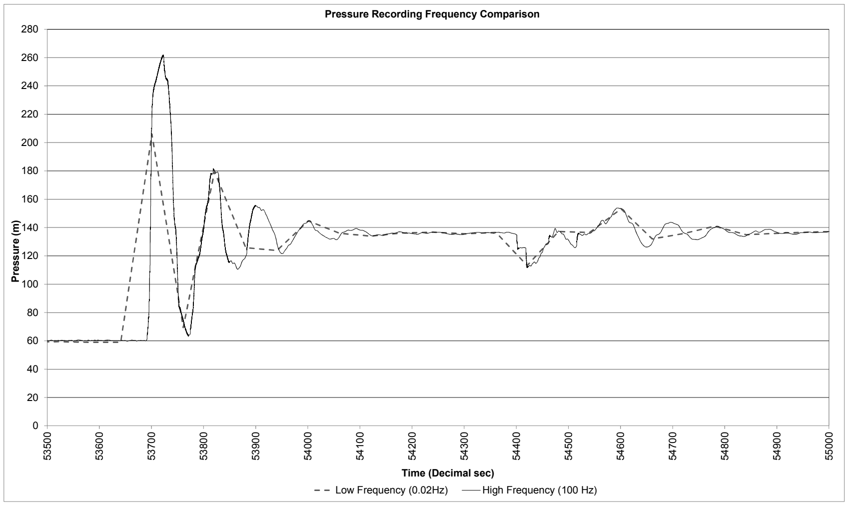

Figure 7: Low and high frequency logging at the same site

Simultaneous low frequency (1 Hz) and very high frequency (up to 1000 Hz) recordings were captured at the same locations along the pipeline routes. The objective was to determine the sensitivity of the recorded data in relation to the frequency at which the data was recorded. It was found that for this pipeline, high frequency logging was not essential to provide accurate details of the high pressure spikes. It should however be noted that the rate of the pressure change, the length of the pipeline and the wave celerity will influence the minimum required frequency for data collection. Every pipeline should therefore be considered on its own merits.

On the day of the assessment, a range of operating scenarios (i.e. valve operations) was executed.

The combined flow log at the outlet of Davel Reservoir and the inlet of Kriel Reservoir is illustrated by Figure 8. The comparative flow rates indicate a pressure dependent behavior i.e. under lower flow/higher pressure conditions, the relative flow rates differ more while under higher flow/lower pressure conditions, the relative difference in flow rate is less. The flow measurements therefore indicate that there was some water loss between Davel and Kriel during the time of logging, attributable to open or leaking off-takes and pipeline leaks.

The combined pressure logs on the Davel-Kriel pipeline is illustrated by Figure 9. The following was observed:

- The ring needle valve is very effective in controlling the flow and preventing excessive surges on opening and closing.
- The final closure of the valve is followed by a period of mass oscillation of pressure waves through the system. The relatively slow rate of decay is

indicative of a rigid system that does not absorb a lot of energy through expansion of the pipe wall, typical of rigid PCP.

- Approximately 20 min after full closure, an offtake was re-opened, resulting in an unexpected pressure transient.

The combined pressure and flow log at the inlet to Kriel Reservoir is illustrated by Figure 10. The inter-relationship between pressure and flow is evident.

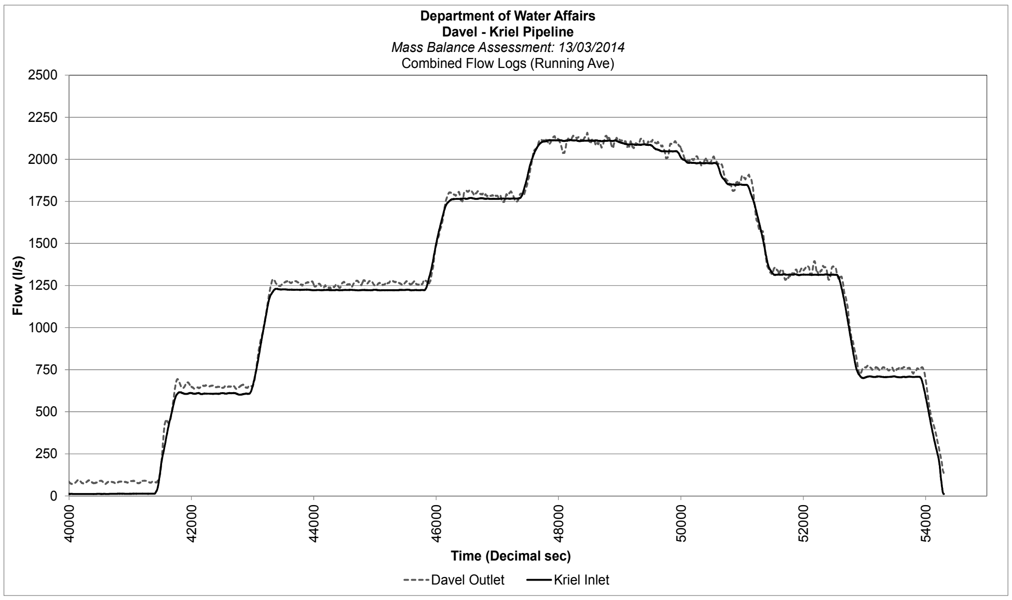

Figure 8: Davel-Kriel combined flow log

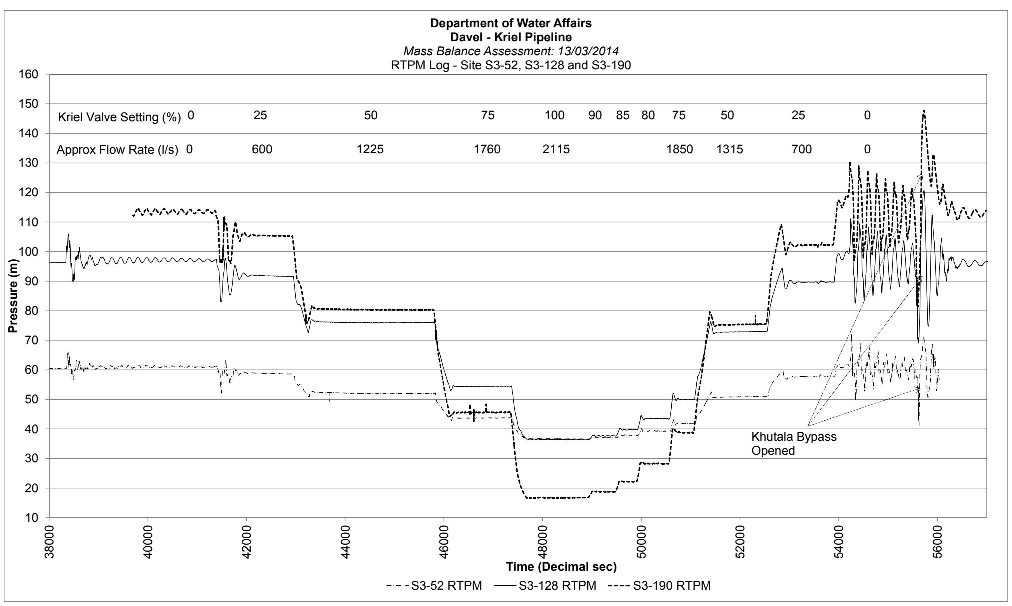

Figure 9: Davel-Kriel combined pressure log

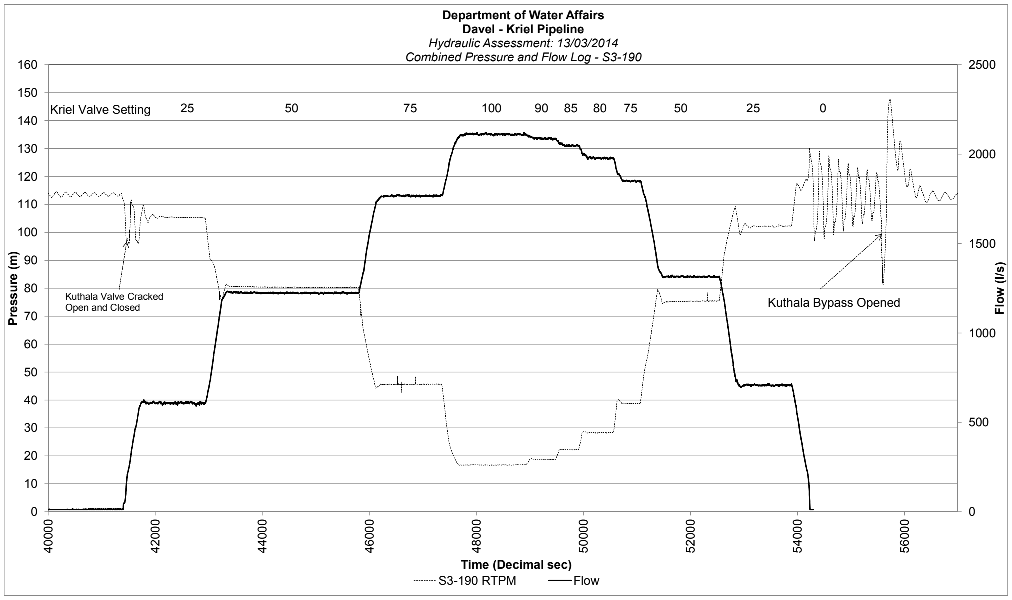

Figure 10: Davel-Kriel combined pressure and flow log

PIPELINE PARAMETER CONFIRMATION

The data gathered during the field verification was used to confirm the following system parameters for inclusion in the hydraulic model:

- Pipeline roughness parameter,
- Wave celerity, and;
- Ring needle valve characteristics.

Pipe Roughness Parameter

The modelled and measured Hydraulic Grade Lines (HGL) was compared at different flow rates along the pipeline route under steady state conditions to determine the roughness parameter. Very good correlation between the measured and modelled values was achieved at an absolute roughness (k_s) value of 0.8mm for this pipe section. The modelled and measured HGL at different flow rates is illustrated by Figure 11.

Wave Celerity

Pressure loggers at three locations along the route were fitted with GPS' to ensure that all the logged data was time synchronised. The wave celerity of the pipeline was calculated by comparing the passing time of a pressure wave at each site. The values were calculated for different scenarios and averaged. An average celerity value of 1200 m/s was derived in this way for input into the surge model.

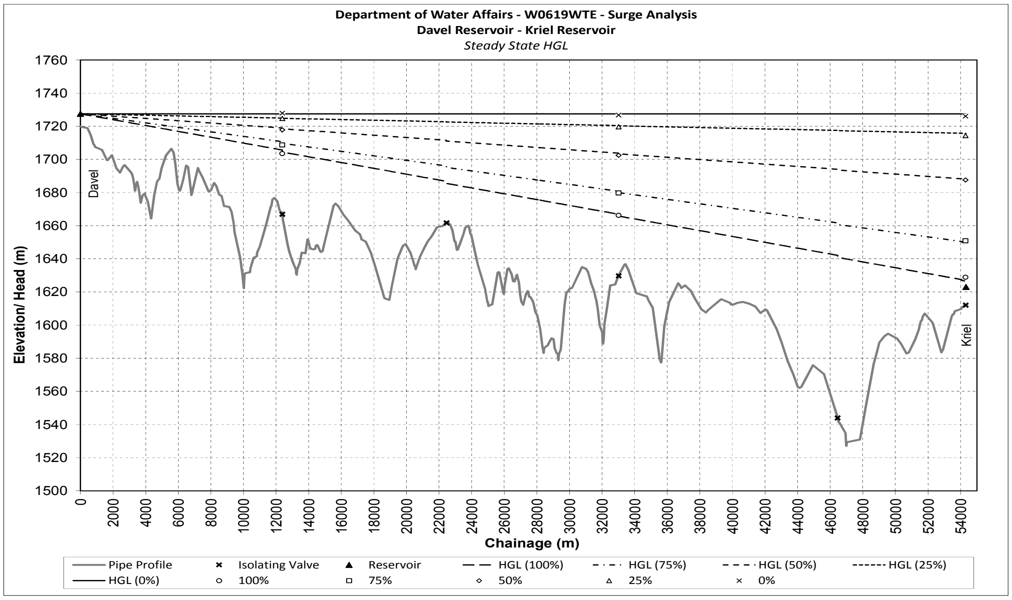

Figure 11: Davel - Kriel – modelled vs. measured HGL

RNV Characteristics

The actual characteristics of the RNV at Kriel inlet had to be determined to ensure that the opening and closing behavior was accurately mimicked in the model. The valve stem vs. area ratio for the valve was determined through trial and error to achieve the actual measured flow rates at specific valve settings as measured during the closing sequence. The resulting valve characteristics are illustrated by Figure 12.

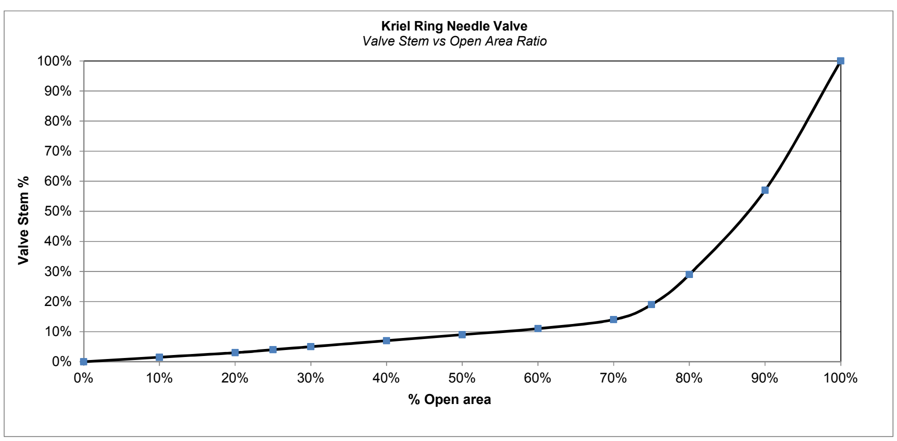

Figure 12: Kriel RNV - valve stem vs. open area ratio

HYDRAULIC MODEL CALIBRATION

Valve opening and closing sequences were analyzed using the Pipe 2012 suite of software developed by KY Pipe and compared to measured results at each logging station. Comparative results at the Kriel Reservoir inlet are illustrated by Figure 13.

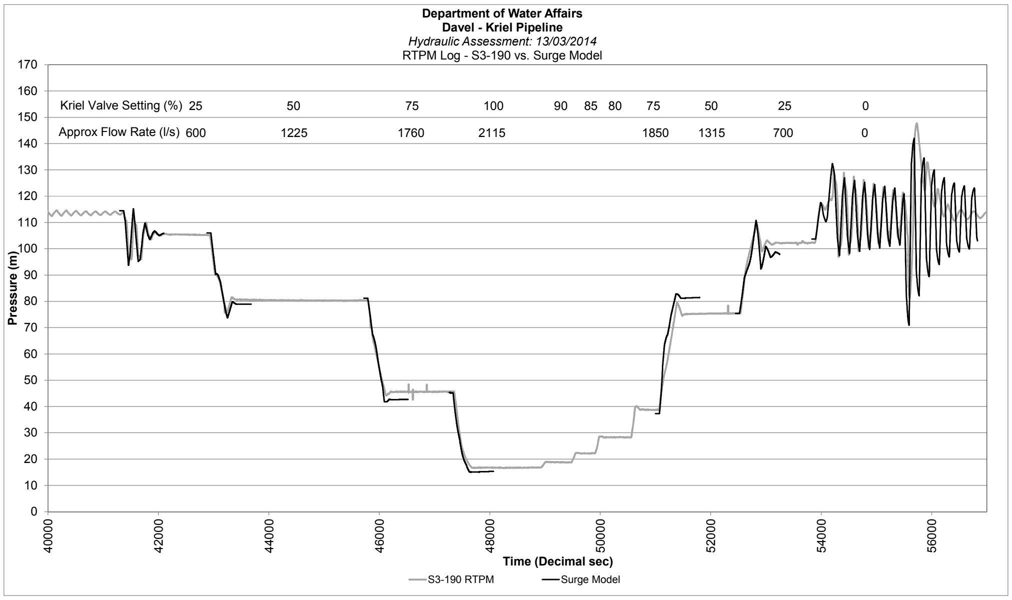

Figure 13: Measured and modelled comparison: Kriel inlet

The following was concluded based on the comparison between measured and modelled results:

- The model mimics the actual system performance under steady state and dynamic conditions during both opening and closing sequences.
- The model accurately predicts the maximum surge pressures during final closure along the majority of the pipeline route.
- Following a complete closure, the model accurately mimics the pipeline period (i.e. mass oscillation frequency) and shows a similar rate of decay to what was measured.
- The opening of the off-take was modelled as a sudden demand on the system. The initial response was accurately mimicked but the actual pressure wave decay differs.

It was found that the model predicted conservatively realistic results for all standard operating scenarios.

SURGE ANALYSIS

The standard opening and closing sequences were modelled and compared to measured results. Complete valve opening and closure results in the minimum and maximum pressure conditions respectively. The combined max/min pressure envelope of all the standard operating scenarios is illustrated by Figure 14. The combined max/min pressure plot is illustrated by Figure 15.

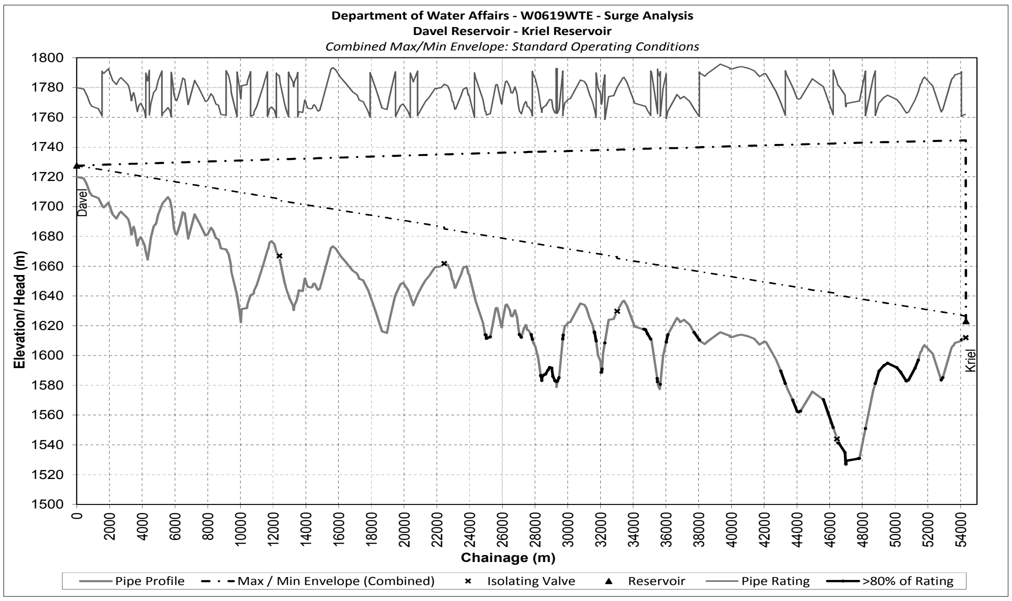

Figure 14: Standard operation combined Max/Min envelope

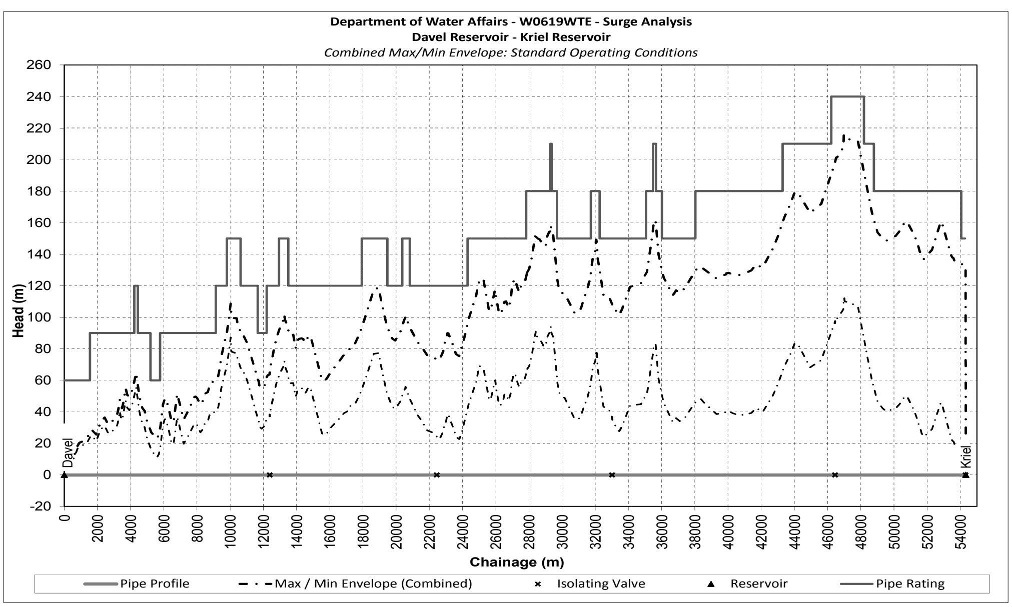

Figure 15: Standard operation combined Max/Min pressure envelope

Although approximately 16% (approx. 8500m) of the pipeline is subjected to maximum pressures that are in excess of 80% of the original designed pipe rating, the surge analysis confirmed that the standard operating procedures were not regularly exposing the pipeline to extreme transient pressures. There was also very little that could be done to further alleviate surge pressures in the system.

The standard operation pressure envelope was incorporated into the Likelihood of Failure (LoF) assessment as part of the engineering analysis of the pipeline.

A number of non-standard operating procedures were also modelled and some were found to produce potentially catastrophic transient events. The calibrated surge model is therefore a valuable tool to illustrate and mitigate potentially hazardous operating procedures before they are implemented in practice.

CONCLUSIONS

The hydraulic model of the Rietspruit-Davel-Kriel pipeline was successfully calibrated using measured pressures and flows along the pipeline route. A number of operational scenarios were modelled to determine its impact on the induced pressure surges. Based on the surge modelling, it was confirmed that the standard operating procedures do not expose the pipeline to unacceptable surges.

The calibration confirmed the accuracy of the hydraulic modelling software and improved confidence in the modelled results.

Hydraulic models compiled as part of the design of any bulk pipeline should be calibrated against measured data to verify that design assumptions were correct and system behavior is accurately mimicked.

Managing Liquid Transients and Vibration within Pump Facilities

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Abstract

Fluid transients causing pressure surge or water hammer are well known to create damaging effects on pipeline systems. It is common for pipeline designers to evaluate transient surge scenarios for their pipeline projects by looking at the pipeline in its entirety from end to end. Any pump facilities located on the pipeline are often considered as a point on the pipeline, with little attention given to the specific piping details of the pump station at a local level. Pressure transients passing through a pump facility can encounter many piping elbows, risers, and piping segments that have small bore attachments for instrumentation, vents, and drains. As the transients pass through the facility, high vibration on the main lines and their associated small bore attachments can lead to fatigue failures with damaging consequences. This paper highlights the importance of considering liquid transient effects on facility vibration for main pipes and small bore attachments using test data from field measured vibrations during known transient events.

INTRODUCTION

In today's world of increasing regulation and serious environmental concerns the incentive for fluid transport companies to minimize the risk of leaks and spills is greater than ever. No industry has felt the pressure of this more than the hydrocarbon pipeline industry in the past few years. With almost daily headlines of pipeline politics and the constant threat of possible leaks and spills within the public eye, there is a pressing need for pipeline companies to analyze and assess the risk of their assets. One prominent risk of leaks and spills is vibration related fatigue failure within pipeline pumping facilities.

Transient response of the piping system due to water hammer events can lead to vibration induced fatigue failures over time. These water hammer events can be caused by pumps starting or stopping, valve swings, transient events from up or down stream of a facility, check valve slam, or operational changes in flow rate. Each transient event has the potential of exciting vibrations of the main pipe, as well as corresponding small bore attachments such as drains, vents, instrument ports, thermal bypass piping, and others. The resulting vibration and stress due to transients can be excessive, as will be shown in this paper.

Water hammer or transient surge analysis within facilities is one area of design where Civil and Mechanical Engineering scopes begin to blur. Often water hammer analysis at the design stage will focus on the pipeline in its entirety, considering a facility as "dot" along the line. The intention of this paper is to shed some light on how a water hammer analysis can be extended toward the mechanical side of this overlap. It highlights some of the transient issues present in facilities, illustrates how the identification and mitigation of these "local risks" is quite different than the transient design analysis typically performed for an entire pipeline, and presents approaches to predict vibration and stress resulting from transient vibrations in pumping facilities.

FATIGUE LIFE, TRANSIENT VIBRATION, AND MEASUREMENT

Fatigue life in steel components is related to its endurance limit, the amplitude of dynamic stress on the component, stress concentration factors, and welding quality. In general, the fatigue life of a component is related to the dynamic stress it experiences. For steel subject to dynamic stresses below the endurance limit, the fatigue life is considered infinite. Any cycles spent at stress levels higher than the endurance limit are said to reduce the components fatigue life (EN 13445).

Fatigue analysis is typically divided into high cycle fatigue (HCF) and low cycle fatigue (LCF). Vibrations and pulsations typically create low amplitude high frequency stresses which lead to HCF. Pressurization, thermal expansions, and transient forces typically create high amplitude low frequency stresses, and lead to LCF.

One problem with transient vibrations is that often a component will only see high dynamic stresses during the transient event itself, which is usually only for a short period of time. This means that each time the transient occurs, a portion of the components fatigue life will be spent. In this way components that have functioned well for many years can suddenly fail without warning once their fatigue life is finally spent.

Another problem with water hammer (transient) induced vibrations is they often do not last long enough for anyone to notice. For these reasons transient vibrations can be thought of as a "silent killer".

Figure 1. Example of data collection equipment required to measure transients' vibrations on hundreds of simultaneous test points

In order to measure and evaluate transient vibrations, you need to be at the right place at the right time or you will miss it. For that, an advanced understanding of water hammer transients is required. Also, operations limited ability to perform transient test events adds to the complexity of their identification and evaluation;] so even if you want to test and measure it, there are limited opportunities to do so.

Figure 1 shows a temporary instrumentation setup to measure transient vibrations at hundreds of test points simultaneously at a pipeline pump facility. A setup like this allows for the collection of many points at facilities with limited transient test opportunities.

VIBRATION EFFECTS OF TRANSIENTS ON PUMP FACILITY PIPING

Generally water hammer analysis is concerned with transient induced pressure surges causing over or under-pressure of pipelines and piping systems. An additional concern for facilities is how water hammer transients will make the piping structures move. This movement, or vibration, is caused by unbalanced forces created in spans of pipe between elbows, and changes in pipe size, as pressure surges pass by. A simple example of this is shown in Figure 2 (top), where a travelling pressure wave between two elbows has a peak-to-peak pressure differential of 60 psi. This causes an unbalanced dynamic force toward to the right side of the diagram, which can be calculated for a 6" pipe with area about 28 in^2 (Force = 60 psi x 28 in^2 = 1,680 lb). As this pressure surge travels by (at the speed of sound of the fluid) the pressures will reverse, causing a similar force in the opposite direction (bottom of Figure 2). This transient force event is applied to the structure to create transient vibration.

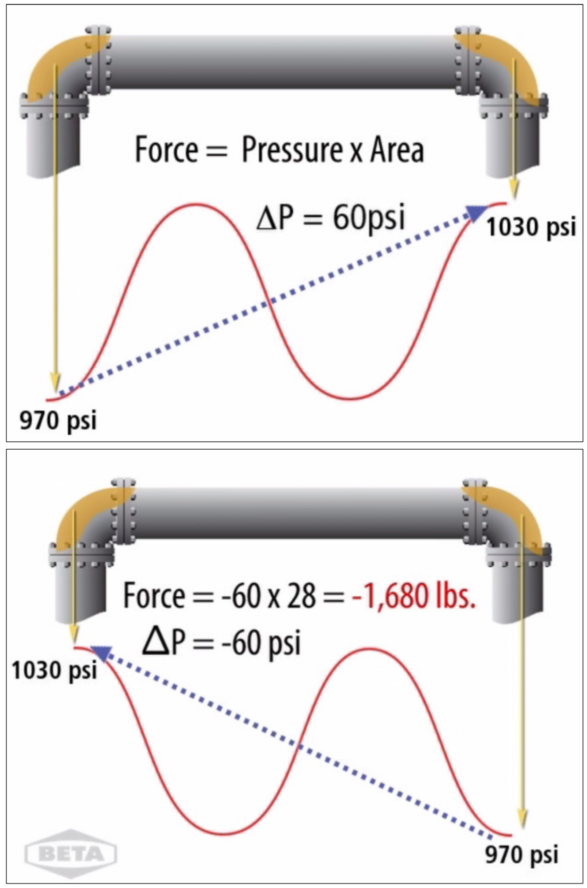

Figure 2. Example of unbalanced force in a piping system

It is possible to determine the transient forces on a given pipe span by calculating the force at each time step of a water hammer calculation (see an example force-time plot in **Error! Reference source not found.**). Some water hammer software's available on the market have this capability.

Discussing dynamic force on its own, however, does not paint a complete picture. Vibration can be described by the following equation:

$$V = \frac{F_d}{K_d} \quad (1)$$

Where:

- F_d = *Dynamic Force*
- K_d = *Dynamic Stiffness*

As such, in order to predict the vibration a dynamic force may cause on the structure, we also need to know the dynamic stiffness of the piping structure.

Calculating the dynamic stiffness of a piping structure can be a complicated task that often involves finite element modeling. It is therefore desirable to design a system with a transient force guideline to limit the amount of force a system will see. The idea is that by keeping transient forces low (F_d in Eq.1), transient vibrations will also be kept low. Transient force guidelines in the industry are rare, but one can be found in the Energy Institute standard "Guidelines for the Avoidance of Vibration Induced Fatigue Failure in Process Pipework" (Section T2.8.3.3).

In cases where high transient forces exist, but suitable modifications to reduce these forces are not practical, transient vibration and stress of the piping system must be calculated to evaluate the severity of the issue.

Applying time varying forces to calculate vibration in structures is not a simple task; it involves a combination of water hammer analysis to produce transient forces, and finite element software to determine the dynamic stiffnesses. Combining these two pieces into a forced response analysis can produce predictions of vibration and stress on a particular component. Many approaches are available to do this; one specific example using common commercially available software packages was given by Wilcox and Walters (2012).

An example of how high vibration levels can be on main pipes subjected to water hammer transients is shown in Figure 3. This vibration data was collected on an 18” riser going into a 36” header during a transient that followed a pump shutdown. It can be seen in the figure that transient vibration levels reached over 6 inches per second 0-peak (over 12 inches per second peak–to-peak) during the event. A common rule of thumb guideline for piping is to keep vibration below 1 inch per second 0-peak.

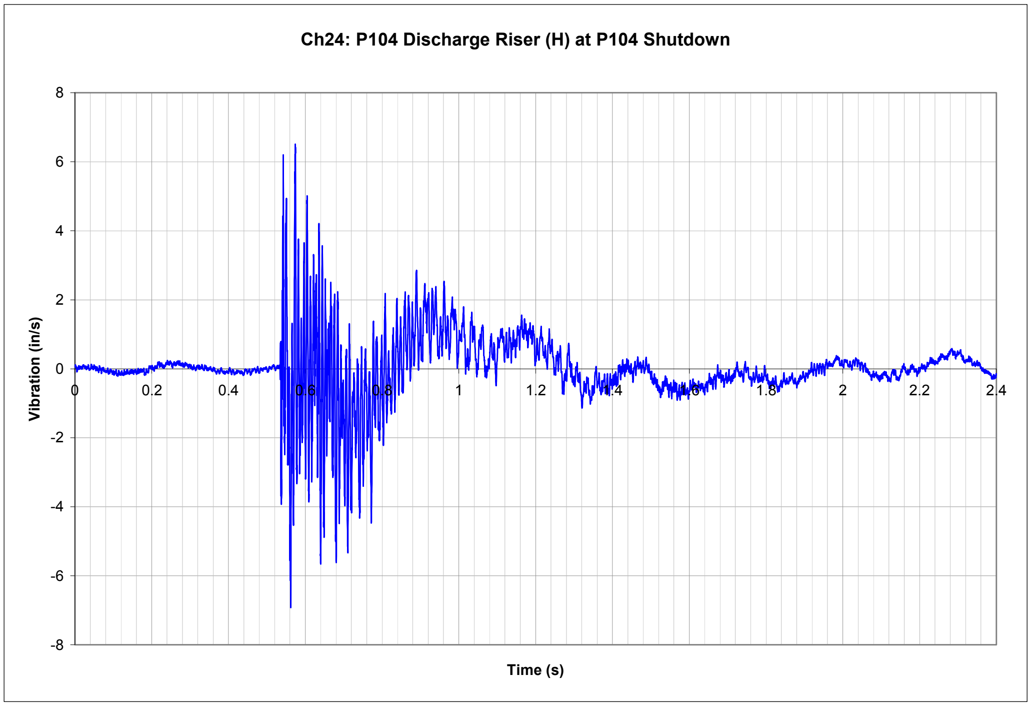

Figure 3. Transient piping vibration of a discharge pump riser after pump shutdown

Transient vibration levels such as those shown in Figure 3 must be evaluated for risk of high stress, fatigue life, and ultimately failure. If found to be high, solutions can involve reducing water hammer transient forces, increasing dynamic stiffness, or both.

VIBRATION EFFECTS OF TRANSIENTS ON SMALL BORE PIPING

Another high risk area for leaks and spills to occur in a pipeline pump facility is small bore piping. The previous section described how transient vibrations are created from water hammer events on main piping, this section describes problems and considerations for small bore connections (SBCs) to the main pipe.

An SBC is defined as a branch connection on the mainline piping that is NPS 2” and smaller. For larger bore pipes (above 24”), connections of up to NPS 4” are also considered to be SBC’s. They come in the form of instrumentation ports, vents, drains, inspection ports, and more. Some examples are shown in Figure 4.

Figure 4. Examples of small bore attachments in pipeline systems.

As described by Harper (2014), it is rare to have design specifications requiring SBC vibration audits during the design stage, or even during field commissioning. Most specifications occur on P&ID’s or isometric drawings without suitable design details. As such, it is left to the field installers’ whim to decide what the SBC design will be. It is thus common at pump facilities to see every SBC on a site being different. This makes an assessment of transient vibration risks very difficult. Given this, the author has witnessed some pipeline operators beginning to standardize its SBC designs in the wake of many costly failures and challenges mitigating the risks.

In considering SBC transient vibration we must be aware of the main line pipe the SBC is attached to. When the main line pipe is subjected to transient vibrations, anything attached to it is also subjected to those vibrations, including SBC’s. The interesting point with SBC’s however, is that their vibration characteristics operate somewhat independently of the main pipe’s characteristics. For instance, a particular SBC will often have a very different mechanical natural frequency (MNF) than its parent main pipe. And so, even when the main pipe appears not to vibrate significantly, the attached SBC can be quite the opposite.

An example of this is shown in Figure 5, where the blue vibration trace is the small bore vibration, and the red trace is the main pipe vibration. The transient event in this case was a pump startup. Notice how the blue SBC vibration is up to 4 inches per second 0-peak maximum vibration during a transient, while the red main pipe vibration remains relatively low through the transient.

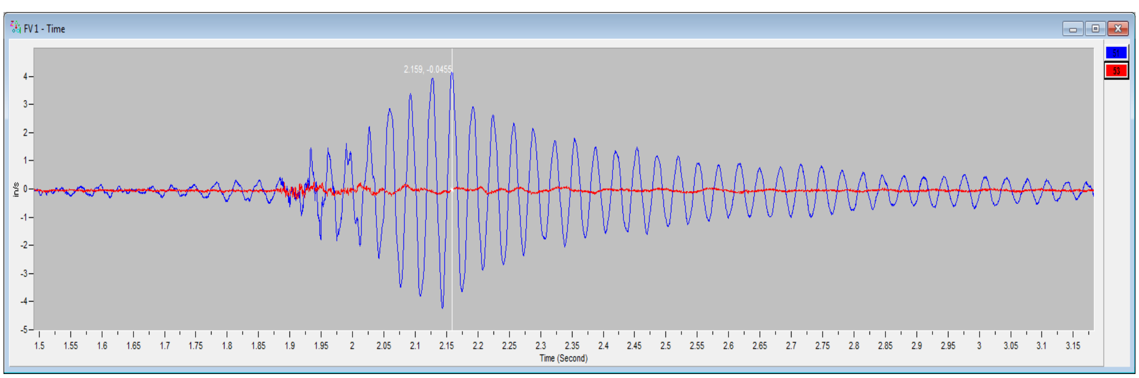

Figure 5. Example transient vibration levels of small bore attachments (blue) compared to main line vibration (red)

Once a vibration level has been measured, it must be compared to a suitable guideline to gauge its severity. Small bore connection guidelines are very sensitive to the layout of the connection and therefore change based on the SBC design. However, using 1 in/s 0-peak as a screening guideline is appropriate in many cases (Harper, 2014). In the case of Figure 6, this vibration was flagged as needing investigation.

The next step is to determine if the measured vibration causes high stress and is at risk of failure. There are several approaches to determine this. One useful method is to calculate an allowable vibration limit given an endurance limit. The results of this method are shown in Figure 6 using finite element analysis (FEA) on the subject SBC of Figure 5. The calculation determined an allowable vibration for this particular small bore of 2.1 inches per second 0-peak. It should be noted that this is a high cycle fatigue analysis, and the allowable vibration limit of 2.1 in/s 0-peak will ensure an infinite life of the component. Figure 5 clearly shows that this particular SBC is at risk of HCF failure.

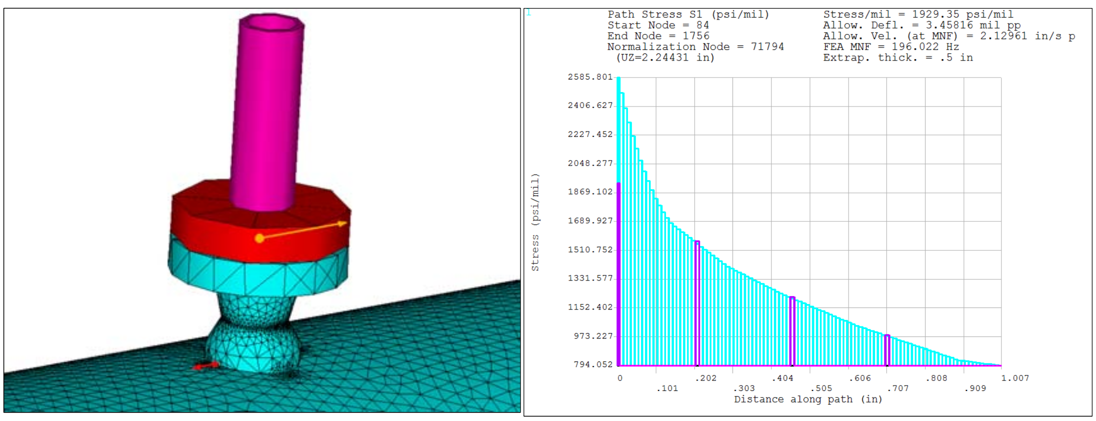

Figure 6. FEA model of small bore attachment with resulting allowable vibration calculations

The analysis can be continued with a LCF analysis to determine the number of cycles to failure, and a prediction can be made of the time, or of how many transient events this particular SBC can survive before failure. In the case of the above example, the client opted to modify the SBC to prevent the high transient vibration in the future.

SUMMARY

Water hammer transients in pumping facilities entail many failure risks. Consequences of these risks are magnified in pipelines transporting hazardous materials such as hydrocarbons which are under increasing regulatory pressure to minimize loss of containment.

Facilities have special considerations of water hammer effects that are not normally considered by analyses used to design an entire pipeline. These “local” facility considerations include:

1. Transient vibrations of main pipes induced by water hammer events. These vibrations are influenced by:
    a. Transient water hammer dynamic unbalanced forces in pipe spans

b. Pipe system dynamic stiffness

Both can be combined in a forced response analysis to calculate resulting vibration and stress.

2. Transient vibrations of small bore connections by water hammer events. Small bore risks are influenced by:
    a. Availability of SBC design specifications
    b. Installation practices
    c. SBC mechanical natural frequencies, and their interactions with water hammer unbalanced forces

Including these considerations in a pump station design will significantly reduce the risk of failure, leading to reduced incidences of leaks, spills, and significant loss of containment of transport fluids.

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EN 13445-3 Unfired pressure vessels - Part 3: Design, July 2009

The Need for Holistic System-Wide Transient Assessment

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Abstract

Water managers are becoming more aware of the impacts of hydraulic transients on their systems in terms of structural integrity, water quality and operations. In the past, transient analysis was typically completed as part of individual pumping or pipeline works design. Distribution systems have likely evolved considerably since transient protection equipment was designed and constructed, in terms of water demands, infrastructure expansion and operations. A comprehensive system-wide transient assessment provides managers and operations with a better picture of the transient impacts of both routine and severe operations at pumping facilities and within the distribution system. It facilitates more intelligent decision making as to the best transient management practices. Case studies of system-wide assessments for several medium sized municipalities are presented. These municipalities desired a review of existing protection and operations of the systems in their entirety. The studies critically evaluated protection effectiveness and whether they benefit or are detrimental to the system. This evaluation determined the criticality and benchmarking of protection for all facilities and evaluated protection enhancements and maintenance priorities. Operational and process control procedures were evaluated from a transient standpoint. This led to the development of a transient management strategy for the system as a whole from both a planning and operations perspective, including short and long term best practices related to transients.

INTRODUCTION

Hydraulic transients occur when a pipeline system changes from one steady state condition to another. Transients are inevitable and occur to some degree in all water systems. Causes can range from routine pump or valve operation to more severe pump trips or rapid valve changes. These can result in pressure fluctuations ranging from smooth, low magnitude changes to rapid and severe pressure instabilities. Often, the primary transient event can result in severe secondary transients (e.g. vapour cavity formation and collapse).

Transients present a variety of threats to the system, including impacts to quality of the delivered water and structural integrity of water system components. These also play a major role in system operations, including procedures and maintenance requirements. It is therefore imperative that water system operators and managers clearly understand the nature and impacts of transients on their systems.

This paper establishes the need for a comprehensive system-wide transient evaluation and outlines study objectives and outcomes. This evaluation will lead to the development of an effective transient management strategy, including protection, transient friendly operating procedures, inspection and maintenance priorities and improved system understanding.

TRANSIENT ISSUES AND MANAGEMENT GOALS

There are numerous issues related to transients within water systems. Typical examples include:

- How are transients generated and how does our system respond to them?
- What areas of the system are susceptible to negative pressures transients?
- Are transients negatively impacting water quality within the system?
- How well protected is each pressure zone relative to other zones?
- How well maintained is existing transient protection equipment?
- Why is one zone more susceptible to transient related breaks?
- How are current operations contributing to transient problems within the system?
- Where should we prioritize system upgrades and rehabilitation to decrease or eliminate transient related water quality risks?
- Does existing protection actually protect the system or make transients worse?
- Which of our 500 air valves should we be maintaining more frequently?
- What is the impact of future flows on transients within existing feedermains?
- How well educated are Operations staff in transient issues?

These issues can be categorized within three overall priorities for transient management: 1) Maintain structural integrity – control extreme pressure variations and lower magnitude cyclic loading that can damage piping and equipment; 2) Protect the quality of the delivered water – reduce the potential for intrusion contamination under negative pressures through pipe wall perforations, faulty joints and air valve chambers as well as reduce extreme velocity changes that can loosen biofilms from the pipe walls; and 3) Improve operations – Ensure controlled valve and pump operations, reduce breaks and long term wear on system components, improve management of air within pipelines, reduce leakage and energy costs, prioritize inspection and maintenance of protection equipment and improve hydraulic performance.

Traditional transient analysis approaches ranges from no transient analysis at all, simplified ‘rule of thumb’ analysis and ‘forensic’ analysis following recurring problems or failure. Additionally, most transient studies are typically completed as part of the design process for individual new or upgraded pumping station or pipeline projects. Recommendations are incorporated into the works being designed in terms of surge protection devices and pipe class requirements. The scope is often limited to the specific works under design, often in isolation of the overall system. These typically focus on the perceived 'worst case' transient events (e.g. global power failure).

A common misconception about transient modeling is that it is only required for the isolated pumping station and pipe system under design, with the assumption that it is more conservative to exclude the connected local watermain systems, as these will dissipate transients. This ignores the transient impact of the proposed works on the local systems, which may be highly susceptible to groundwater intrusion and pipe breaks. It also neglects the potentially severe primary or secondary transients that can travel from the local system to the proposed works. Some of these traditional approaches can result in surprises at the design or post commissioning stage, inappropriate, oversized or undersized transient protection and hydraulically inefficient pipeline profiles. These approaches fail to treat the system as a whole and can lead to a piecemeal approach to transient management within the system, which can result in a detrimental impact of the proposed works or protection on other areas of the system.

There is often a disconnect between the initial design level analysis and actual long term system operations. Water systems continually evolve over time since the works were designed, including system expansion, grid reinforcement, changes in water usage, rehabilitation, degradation, changes in leakage as well as operational changes. Therefore it is critical to determine whether the previous transient analysis assumptions and recommendations and current operations and protection are still appropriate for today's conditions.

A MASTER PLAN APPROACH TO TRANSIENT PLANNING

The limitations of traditional transient analysis highlights the need for a system-wide or 'master plan' approach to transient planning to improve the understanding of the implications of transients on the water system as a whole. Several papers emphasize the need for comprehensive transient analysis and document limitations and caveats of 'traditional' approaches. (Karney, 1990), (Jung, 2007). Benefits of a comprehensive system-wide analysis include:

a) *Analyzes the system as a whole* - Most municipalities have developed detailed all-pipe hydraulic and water quality models of their systems. These are directly compatible with sophisticated and computationally efficient transient modeling software packages and can be readily leveraged for system-wide transient model evaluations. The models can be used to assess a wide range of transient events, system conditions and protection. This can identify transient interactions between individual systems within the network such as between multiple pumping stations servicing the same zone or between large diameter feedermains and local mains.
b) *Defines the many transient issues within the entire system using a common benchmark* - This facilitates a 'big picture' overview of transient issues and risk on a system-wide scale. This approach is a good opportunity to involve operations through workshops and field reviews to educate operators on transients, determine how the system is operated and gain operator insights into specific system issues.

c) *Provides a consistent, systematic approach to transient evaluation and management* - A transient master plan enables efficient evaluation of areas of concern within the system and mitigative measures. This provides a 'Best Practices' approach to transient management across the system. Key action items are facilitated, such as establishing targets for design, developing system-wide transient models, critical evaluation and critique of protection methodologies, inventory and inspection of protection assets, review of protection equipment design and maintenance, risk assessment, water quality sampling, development of a common approach for transient modeling, analysis and design, operator education, establishing common operating procedures for transient prevention and development of a holistic, yet cost-effective transient management program.

d) *Part of an integrated approach to overall water system planning, engineering and operations* - Transient analysis results can be integrated with other facets of water system planning and engineering, including steady state hydraulic modeling, water quality sampling and modeling, master servicing plan, maintenance management system, condition assessments, rehabilitation / replacement needs and operational protocols. This approach presents opportunities to leverage the use of other data to integrate and correlate results of separate studies, information or programs and phase major protection works with planned system upgrades.

A system-wide transient analysis is essential to achieve a comprehensive understanding of the system and a proactive approach to management and operations.

CASE STUDY EXAMPLES OF SYSTEM-WIDE TRANSIENT ANALYSIS

Case study examples of system-wide transient analysis applications are presented for two Ontario, Canada water systems ranging from 350,000 to 1.1 million serviced population. Each of these systems had detailed and calibrated water network models. These examples are presented in terms of how they addressed the major priorities for transient management, including water quality, structural integrity and operations.

Protect Water Quality – Recent studies demonstrate that given appropriate conditions, there is a potential for degradation of water quality within the distribution system as a result of transient conditions (LeChevallier, 2003). System-wide analysis can be used to identify portions of the water distribution network that are vulnerable to water quality risk. Potential mechanisms affecting water quality are discussed in this section.

In all the case studies, the potential for groundwater intrusion into watermains through pipe wall perforations or faulty joints under negative pressure transients was assessed. A generalized pipe condition rating was determined for all mains based on pipe age and material. Mains with a condition rating of 3 or 4 consist of older cast or ductile iron that are assumed to be susceptible to transient related deterioration and groundwater intrusion. Mains with a rating of 1 or 2 are assumed to be substantially watertight, such as PVC, or newer concrete pressure pipe.

Mains subject to negative transient pressures based on the modeling were correlated with poor condition pipes. This is shown graphically on Figure 1 for a portion of the system. Based on the transient modeling results, the aggregated length of main susceptible to negative pressure transients for each condition rating was estimated. This was completed for existing as well as improved transient protection to evaluate its effectiveness on potential groundwater intrusion reduction within vulnerable areas.

The risk for standing water intake and pathogen intrusion to the watermain from flooded air / vacuum relief valve chambers under vacuum relief (negative pressure) conditions was also assessed in the case studies. In one of these, sampling and chemical analysis of standing water within air valve chambers indicated results approximately equivalent to first flush surface runoff consisting of aerobic endospores, Total coliforms, E. coli, C. Perfringens and Enterococci. One of the case studies included a detailed inventory and inspection of all air valve chambers in the system. This identified chambers that are subject to inflow/infiltration through the chamber walls or cover. Many chambers had a high water mark above the air valve intake. Along with this, the transient model results identified surge critical air valves, or those that would activate and provide vacuum relief during a transient event to control negative pressures. These surge critical air valves were then correlated with chambers having a high risk of flooding. Flood prone chambers with risk of intrusion of standing water under vacuum relief operation were flagged for rehabilitation, increased inspection and maintenance.

A potential transient related water quality risk is rapid flow fluctuations or reversals following severe transients. This has the potential to shear biofilm or corrosion by-products from pipe walls and / or re-suspend sediments within the pipe. One of the studies reviewed and compared model outputs for steady state and maximum transient velocity as well as the maximum flow reversal velocity for all the modeled pipes. Vulnerable areas for scouring were identified by correlating these results with the pipe condition rating, which is assumed to be indicative of biofilm growth and corrosion susceptibility. Mains having a low steady state velocity and transient velocity above the resuspension velocity were also flagged. These areas should be prioritized for increased flushing and sampling following power failure events, with corrective action as necessary.

The transient water quality assessment demonstrated the need for being proactive in reducing the risk for contamination and developing best practices for transient management. Water quality risk areas were flagged for prioritized watermain replacement/rehabilitation, transient pressure monitoring and improved protection.

Maintain Structural Integrity - System-wide transient analysis results can be used to identify portions of the water distribution system that are vulnerable to excessively high or low transient pressures due to either routine or abnormal transient events. Many watermain breaks are transient related (directly or indirectly), including high pressure transients causing pipe rupture, negative pressure transients causing joint damage as well as cyclic loading causing long term wear.

For each system analyzed, transient risk to watermain structural integrity was identified by correlating transient model results with the watermain condition rating discussed previously. These ratings were correlated with mains that are subject to high transient pressures based on the transient modeling. The aggregated length of main susceptible to high pressure transients for each condition rating was estimated. This was completed for existing as well as improved transient protection to evaluate its effectiveness on reducing transient within vulnerable areas.

For one of the studies, historical watermain break records were spatially correlated with areas of model predicted high transient pressures to determine vulnerable areas. The analysis showed clusters of main breaks that coincided with areas of high upsurge pressures. Many of these clusters were adjacent to pumping stations. The analysis indicated a definite spatial correlation between pipe breakage occurrence and predicted high pressure transients. This analysis was also used to 'validate' model predictions. Further study for this is warranted for these systems. Break patterns should be tracked both before and after transient protection is in place.

The transient structural integrity assessment can be used to develop best practices for transient management, including prioritizing watermain replacement programs and providing input to prioritizing surge protection requirements.

Figure 1: Correlation of Watermain Intrusion Potential with Negative Transient Pressure Potential

Improve Operations - The prevention and mitigation of transients within the distribution system play a major role in water system operations. These can be categorized in terms of operational procedures and transient protection equipment design and operations and were assessed in the system-wide transient case studies.

Operational Procedures – Operating procedures should ensure that day to day operations do not adversely affect the system over the long term. The impact of frequent, low magnitude transient pressures should not be overlooked. Each of the case studies involved operations staff during the study process. Workshops with Engineering and Operations staff enabled operator education through "Transient 101" sessions on transient fundamentals with an expert in the field. These workshops allowed transient thinking to be incorporated into routine system operations to address what effects control actions will have on the system to achieve a better understanding of the consequences and risk of operations activities.

In the case studies, improved standard operating procedures (SOPs) for the system were reviewed for transient prevention and mitigation. Common and consistent guidelines were developed for normal and emergency operations. Examples include smooth startup and shutdown of pumps and valve operation. Following a major transient event such as a global power failure, procedures were reviewed to ensure that the system is brought back on line without detrimental impacts. For example, as shown on Figure 2, the time delay prior to restarting pumps following a pump trip was specified for each station to allow transients to decay, prevent compounding pressure surges and permit exhaust of air valves prior to restarting pumps. Crucial valves in pumping stations were tagged to ensure that they are only operated by qualified personnel. The impact of non-routine operations on transients was also determined. For example, the effect of a pump trip was assessed with a closed valve (normally open) along a long transmission main upstream of a reservoir. As shown on Figure 3, the transient results in a pressure wave reflection at the closed valve, which is now essentially a long dead end, compounding upsurge pressure to above the pipe working pressure. The model allows operators to be more aware of the impact of abnormal operations on transients and effectively plan for them. Guidelines were developed to assess revised operations when critical surge protection is taken off-line or when critical valves or mains are closed.

This approach demonstrates to decision makers the need for increased vigilance at critical locations, formal record keeping, developing best practices and the importance of allocating sufficient budgets for inspection and maintenance.

Transient Protection Equipment Design and Operations - In each study, an inventory of transient protection devices was carried out across the systems to provide a better understanding of the suitability of transient protection equipment. Protection equipment typically consist of surge valves, surge vessels and air valves. For surge valves, inventory data included the type, size, condition, pressure setpoints, as well as a review of maintenance frequency and standard operation procedures.

For many facilities, it is likely that protection device size and setting was established years ago. In the intervening time, operating conditions, demands, pumping equipment, and system hydraulic conditions may have changed. If protection equipment is of an incorrect type, is undersized or has inappropriate settings, it won't adequately protect the system. If it is oversized, it may cause secondary transients during closing. Transient modeling reviewed the adequacy of the surge valve type and sizing, setpoints and criticality for surge protection. Closure characteristics were assessed, including opening/closing time and open duration. For stations equipped with multiple surge valves, primary and secondary valves were identified, with staggered opening and closing settings.

In one of the case studies for a system serviced by multiple stations, surge relief valve benchmarking was completed as shown on Table 1. This benchmarking determined how transient protection compares by station across the system. The analysis compared the relief capacity in terms of existing and proposed pumping capacity, which identified stations that likely have undersized protection. Pressure settings in terms of hydraulic grade line were also compared for each station and assessed relative to the normal pressure range within each pressure zone.

Table 1: Surge Valve Benchmarking Example

Facility Name	Pressure Zone	No. of Surge Valves	Surge Valve Size (mm)	Surge Valve Type	Surge Valve Capacity Evaluation			Surge Valve High Pressure Setting Evaluation			Comment
					Pumping Station Firm Capacity (ML/d) (Future)	Surge Valve Peak Flow (ML/d)	Surge Valve Peak Velocity (m/s)	Pumping Station Operating Pressure (m HGL)	Surge Valve Pressure Setting (m HGL)	Pressure Difference (kPa)	
Z1_PS1_H	1	3	200	Surge anticipator	383	1342	14.2	153.9	161.9	78	Review valve size
Z2_PS1_L	2	5	300	Rate of pressure rise	543	2631	7.4	180.6	189.8	91	
Z2_PS2_L	2	2	300	Rate of pressure rise	360	500	3.5	184.1	189.0	48	
Z3_PS1_L	3	5	300	Rate of pressure rise	1031	297	1.0	217.9	219.8	19	Review valve pressure setting
Z3_PS2_H	3	2	250	Rate of pressure rise	69	700	7.1	222.5	224.1	16	Review valve pressure setting
Z3_PS3_H	3	2	250	Rate of pressure rise	80	432	4.4	198.7	224.9	257	Review valve pressure setting
Z3_PS4_L	3	2	200	Rate of pressure rise	289	484	9.9	221.5	230.4	87	
Z4_PS1_L	4	2	200	Rate of pressure rise	169	711	11.3	270.0	263.5	-64	Review valve pressure setting
Z4_PS2_L	4	2	250	Rate of pressure rise	297	849	8.6	244.3	253.5	91	
Z4_PS3_H	4	2	250	Rate of pressure rise	107	460	4.7	234.1	258.5	240	Review valve pressure setting

Air valves provide air release and vacuum relief protection for watermains operating under fill / drain, main break and transient conditions. Improperly sized or maintained air valves won't provide the intended protection and can worsen transients. One of the case study systems has over 300 air valves across the network, resulting in a maintenance burden for the operators. Therefore the City needed a means of determining which air valves are critical for operations and transients to prioritize maintenance and replacement. The air valve transient criticality assessment involved detailed inspection and inventory of all valves in the system. Detailed transient modeling was conducted to assess transient performance of all the air valves to determine their role in providing transient protection and operational functionality. This defined the criticality of each air valve on a scale of 1 to 4 as presented on

Figure 4. Surge critical air valves were defined as those that provide primary or secondary surge protection. This provided a more manageable list of key air valves based on criticality for prioritized inspection and maintenance.

Surge critical air valves were also reviewed in terms of their type and sizing. This identified air release only or manual valves that should be retrofitted with vacuum relief protection for transient control. Many of the valves were found to be oversized, resulting in pressure spikes when exhausting the air. Figure 5 shows the effect on air valve pressure following a pump trip with replacement of an existing 150mm standard air and vacuum valve with a 50mm combination air valve equipped with surge suppression (or non-slam) orifice. The existing valve results in high pressure spikes when the final air volume is expelled. The replaced valve provides the same negative pressure protection, however exhausts the air at a slower rate, resulting in a much more controlled and smooth pressure trace.

Transient modeling was also used to determine transient protection equipment criticality for the system. For example, test runs were completed with a surge relief valve off-line. This type of evaluation can be used to prioritize maintenance to determine the transient response with critical transient protection off-line. It can also provide information on how operations should be changed for planned surge protection maintenance as well as testing protection redundancy with primary protection fully or partially off-line.

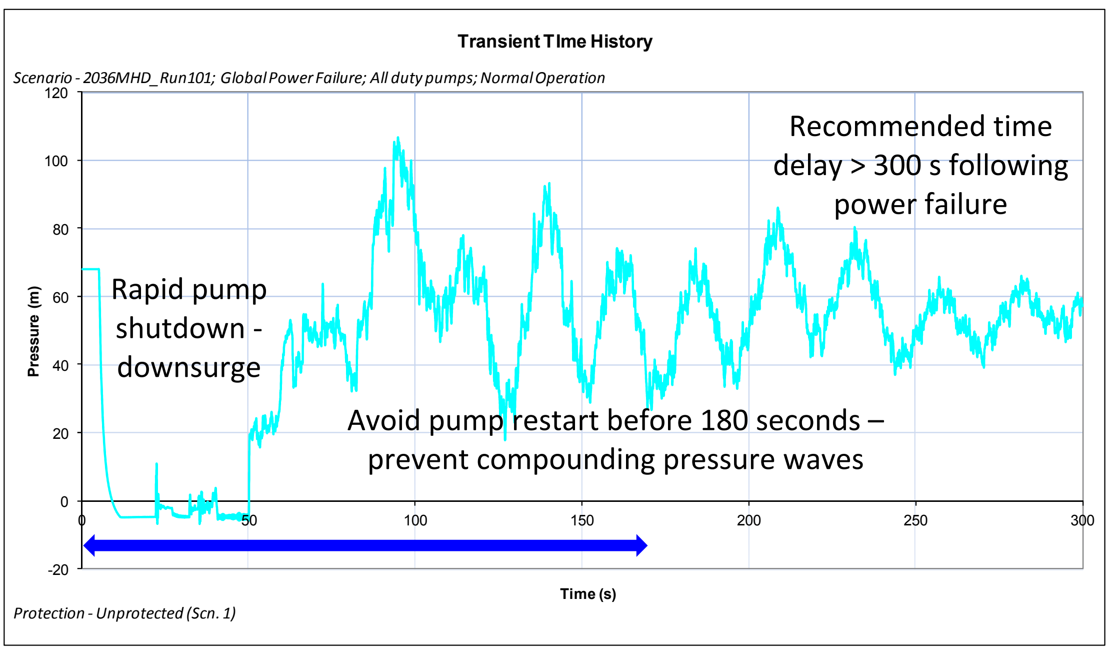

Figure 2: Pump Station Discharge Pressure Following Pump Trip

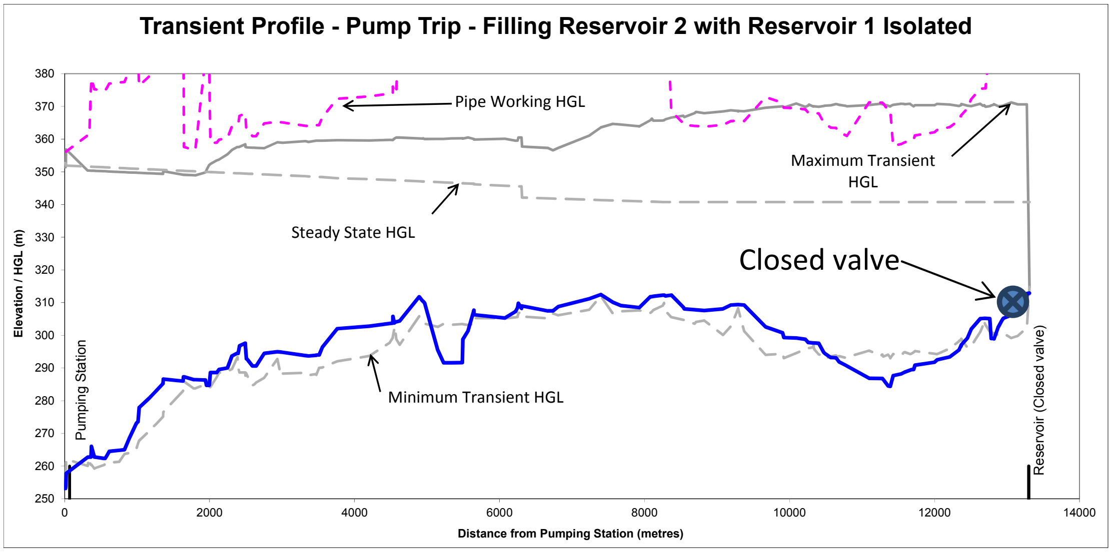

Figure 3: Transient Effect of Pump Trip with Transmission Main Valve Closed

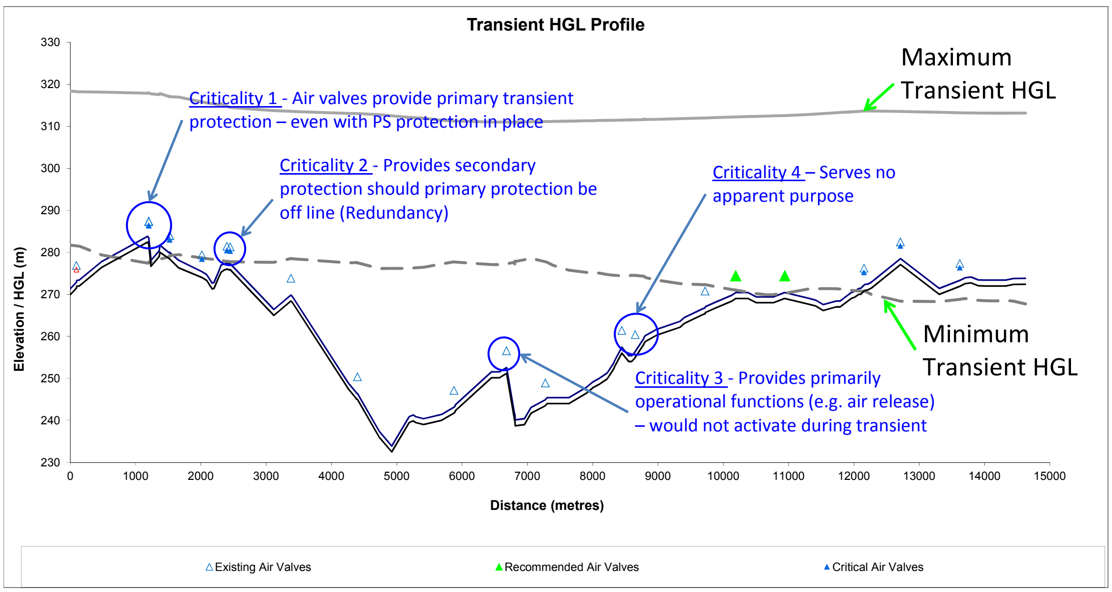

Figure 4: Air Valve Transient Criticality Assessment

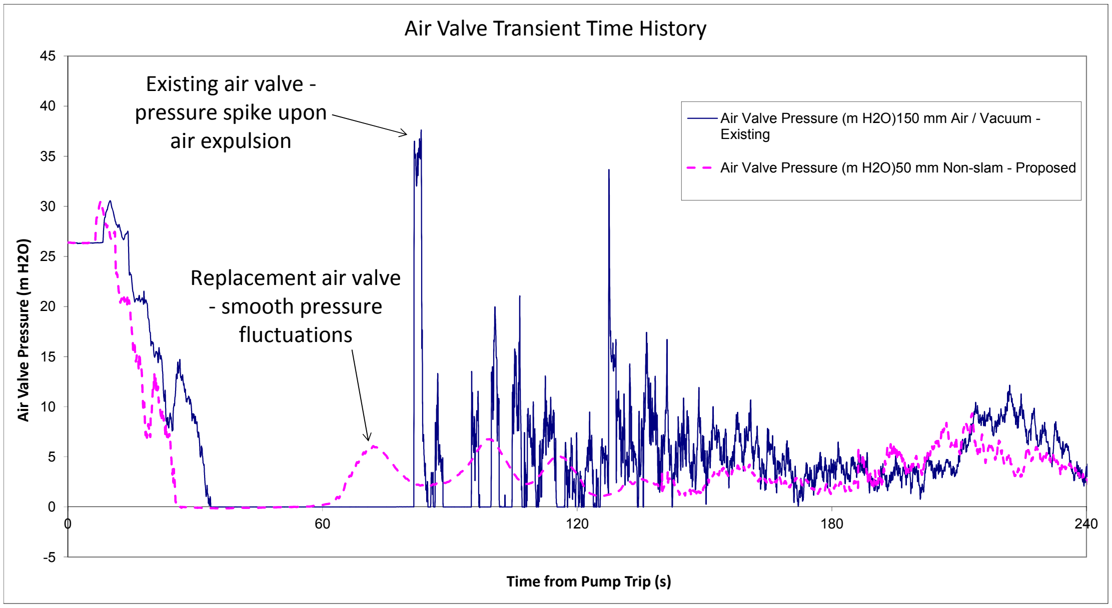

Figure 5: Air Valve Transient Pressure Following Pump Trip – Air Valve Type and Size Comparison

DEVELOPMENT OF WATER SYSTEM TRANSIENT BEST PRACTICES

Each of the case studies developed water system best practices recommendations related to transient management. Short term best practices include:

- Create system-wide transient model of system using updated and calibrated network model.
- Use model to evaluate existing and required transient protection equipment.
- Use model to analyze current and improved operations.
- Correlate results of watermain condition ratings with predicted high transient pressures to determine areas at risk for structural degradation and main breaks.
- Correlate results of leakage studies and watermain condition ratings with predicted areas vulnerable to negative pressure transients to determine areas at risk of water quality contamination.
- Correlate vulnerable areas with water quality modeling results for predicted chlorine residual or water age to ensure quality maintained at these locations and determine if additional disinfection is warranted.
- Identify and evaluate appropriate operations response to major transient events such as targeted water quality sampling within vulnerable areas.
- Survey, inspect and inventory air valve chambers.
- Use modeling to determine surge-critical air valves.
- Identify flood-prone air valve chambers subject to negative transient pressures and water quality risk.
- Develop improved air valve inspection, maintenance and rehabilitation plan, prioritizing critical valves.

- Develop design standards for new and retrofitted air valve chambers to control inflow/infiltration. Modify air valve venting design to vent air externally or as high within the chamber as possible, combined with an inflow prevention device.
- Prioritize maintenance of transient protection devices based on criticality.
- Conduct transient pressure monitoring and testing to provide data for transient performance and transient model validation.

Long term best practices include:

- Implement new or enhanced transient protection at pumping stations and distribution system to mitigate high and low pressure transients.
- Implement air valve chamber rehabilitation program.
- Include transient water quality and structural vulnerability assessments in watermain rehabilitation and replacement prioritization.
- Enhance water sampling and corrective action within areas vulnerable to transient related ground and surface water intrusion, especially following severe transient events, to assess contamination risk and ensure adequate chlorine residuals are provided.
- Incorporate transient modeling to test system response and plan operations under both routine and non-routine operations, e.g. determine the impact of facility off-line scenarios such as taking a trunk main off-line or temporary closure of an elevated tank.
- Use transient modeling to determine system operational limitations with transient protection off-line.
- Recommended operational procedures should be incorporated into the process control narrative, control logic and operations manuals of major facilities.

CONCLUSIONS

System-wide transient analysis is essential to achieve a comprehensive understanding municipal water distribution systems. It provides a proactive approach to system management and operations by reviewing the transient impacts on operations, structural integrity and water quality. Analysis results can be utilized to review and benchmark transient protection across the system and prioritize maintenance of protection devices based on criticality. Results can identify areas of the system vulnerable to water quality and structural degradation and diagnose and mitigate areas of chronic transient related watermain breaks and water quality risk. Watermain rehabilitation and replacement and air valve chamber maintenance can then be prioritized accordingly. It can identify appropriate operational responses to major transient events. This will lead to development of an integrated transient management strategy, in terms of protection devices, transient friendly operational procedures and maintenance prioritization, leading to more intelligent watermain replacement / rehabilitation and capital works planning.

Keys to success for a system-wide transient master plan approach include a previous water system master plan, improved hydraulic and transient modeling software, an accurate and up to date hydraulic model and comprehensive system data. It is essential to have proactive engineering and operations staff with a willingness to understand problems, share knowledge and information, buy into different approaches and solutions and be open to modifying operational protocols and transient protection strategies.

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Modeling Halfway Around the World: Advanced Hydraulic Model Calibration for a Large Utility

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Abstract

An extended period calibration was performed for Sydney Water Corporation in Australia, a regional utility which serves 4.6 million people and manages 13,000 miles of water mains. The purpose of the project was to calibrate existing models to maintain an accurate representation of the water distribution system, customer demand patterns, and controls. The calibrated models would then be able to be used by the utility for many purposes including future growth planning, operations optimization, water quality modeling, and incident management. The calibration process consisted of updating models based on the most recent geospatial data of the network assets, updating the demands from the customer usage database, and updating the controls from the SCADA system. The calibration tolerance of +/- 1 m difference between observed and measured hydraulic grade over a 24-hour period was achieved for the majority of calibration points. In order to achieve calibration, the boundary conditions and initial settings were updated based on measured data, the control settings were updated, and the demands were updated to match the demand patterns observed on the calibration day. In some cases additional modifications were needed to achieve the calibration tolerance, such as: pump curve adjustment, valve diameter adjustment, pipe roughness coefficient adjustment, and valve operational status. Identifying these operational issues as a result of the calibration process provided a benefit to the utility and allowed them to focus field efforts on particular areas, such as underperforming pumps and water mains with potential closed valves or other restrictions.

INTRODUCTION

Sydney Water is a regional water, sewer, and storm water utility in New South Wales, Australia. Sydney Water is a large utility, serving 4.6 million people. The service area covers 4,900 square miles and provides 370 million gallons of potable water per day under average conditions. Sydney Water also provides 33 million gallons per day of recycled water for non-potable uses. Sydney Water's water system network consists of the following:

- 7 reservoirs
- 9 water filtration plants
- 1 desalination plant
- 13,000 miles of water pipes
- 251 storage tanks
- 164 water pumping stations

With such an extensive network, Sydney Water has a significant investment in capital projects and water main renewal for maintaining the existing facilities. Therefore, Sydney Water needed to update their existing model to have a decision-making tool for capital investments and renewal programs. Sydney Water desired a model calibrated to a high level, so that it would be suitable for a variety of different purposes, including future growth analysis, temporary shutdown of water mains, pressure rezoning, operational controls changes, water quality analysis, and decommissioning of assets.

APPROACH

In order to achieve the goals for the water model and develop a robust decision-making tool for capital and renewal projects, the following approach was implemented. Refer to Figure 1 for illustration of the project workflow.

1. Obtain necessary field data to supplement the existing pressure, flow, tanks level, and controls monitored by the SCADA system.
2. Update the existing models to reflect current demand conditions and system changes such as pipe and pump upgrades and decommissioning of tanks.
3. Perform Extended Period Simulation (EPS) calibration for the distribution and transmission (trunk) models.

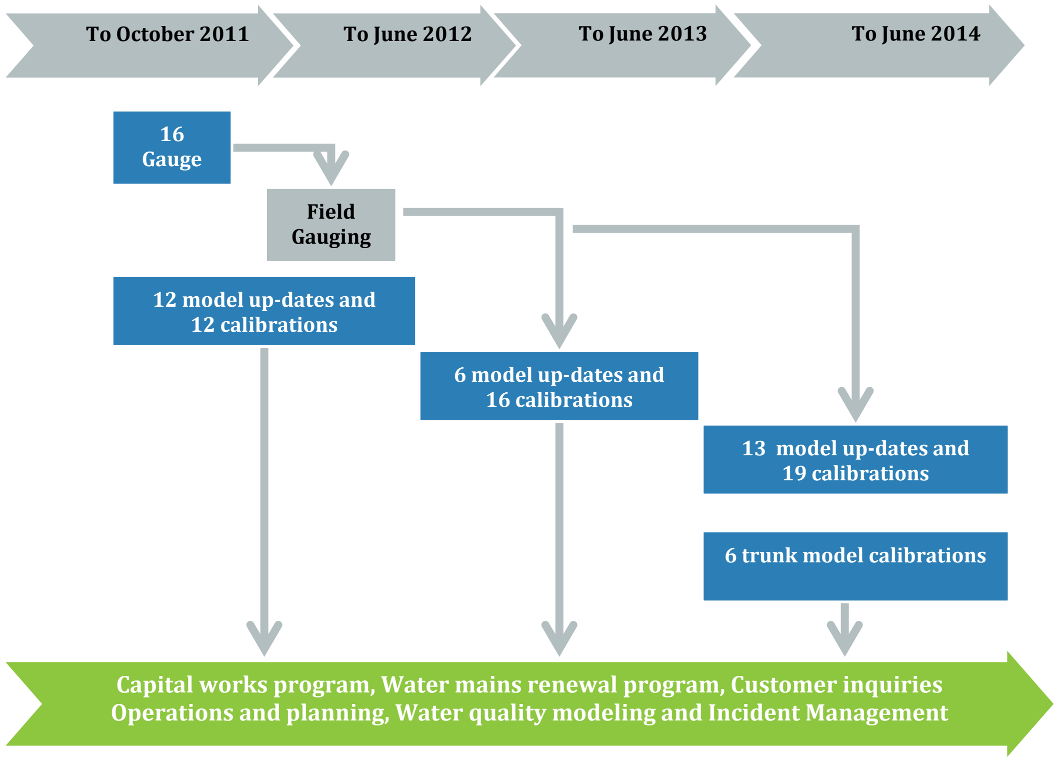

Figure 1. Project Workflow.

INITIAL MODEL UPDATE

Before calibration could take place, several of the models were updated to incorporate recent changes to the distribution system. The model update process included updating the average day scenario with the most recent geospatial data of the network assets, updating the system demand and unaccounted for water, and updating controls. The newly updated average day simulation was compared against typical operating conditions for verification.

CALIBRATION CRITERIA

Once updated for the typical current conditions, the model could be calibrated. The calibration process involved detailed data review, calibration day selection, boundary conditions update, controls update, demand multiplier determination, and demand pattern adjustment.

The calibration criteria used by Sydney Water are as follows:

1. Model Hydraulic Grade Line (HGL) within 2.3 ft (1 m) of measured data.
2. Model flow rates within +/- 2% of the measured data for trunk/transmission mains (15-inch diameter and larger).
3. Model flow rates within +/- 5% of the measured data for distribution mains (<15-inch diameter).
4. Model storage tank levels within +/- 1.2 ft (0.5 m) from the SCADA system.
5. Pump operation matches observed timing from SCADA system.
6. Control valve operation matching measured percent open and timing from SCADA system.
7. Pipe relative roughness Colebrook-White k values, of 0.1 to 3 for water mains up to 24 inches in diameter; and 0.1 to 1 for water mains greater than 24 inches in diameter.

DEMAND ANALYSIS

The customer demands were analyzed to classify demands by category, such as: Commercial, Industrial, Residential (LD) (low density residential), and Residential (HD) (high density residential). An example demand distribution by demand categories is shown in Figure 2. Each demand category represented a diurnal pattern. Analysis of the demands during the gauging period was performed to select the calibration day for Extended Period Simulation (EPS) calibration. The day with the greatest peak hour demand was selected for the calibration day. A calibration day selection graph is shown in Figure 3. The demand multiplier was calculated in order to adjust the calibration day demand for the model analysis.

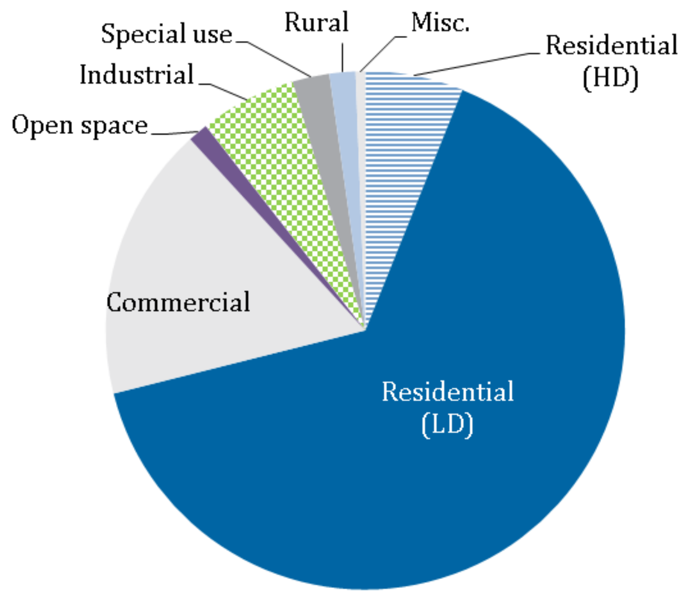

Figure 2. Demand Classification by Category. The residential categories are divided into high density (HD) and low density (LD) properties.

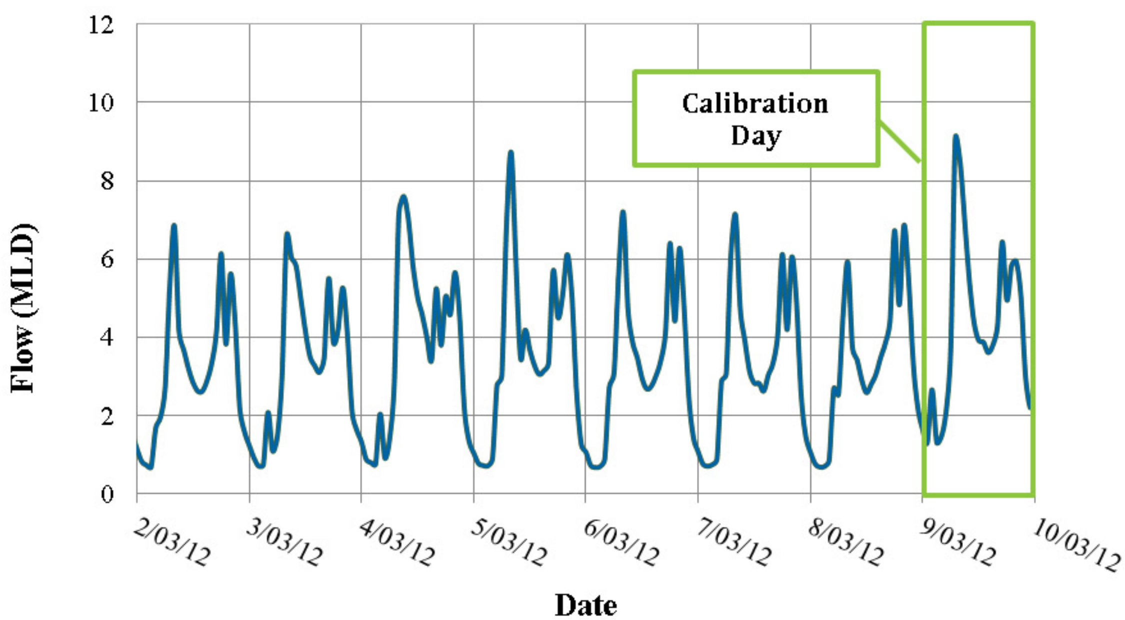

Figure 3. Calibration Day Selection.

DATA VALIDATION

The data collected was reviewed for accuracy, since it would be used to determine the need for model adjustments. Pressure gauges were reviewed for issues such as: instrument accuracy, elevation survey errors, unit errors, installation in another pressure zone, and time sync issues. After data review, some gauges were adjusted based on other results. Figure 4 illustrates an example of a data validation plot, where the HGL of gauges in the same pressure zone is compared. The gauges with inconsistent HGL values during times of low demand are adjusted to match the HGL of other gauges in the zone to account for the gauge error, which could be due to an incorrect elevation.

Flow meter data was also reviewed against total system flow in detail to identify accuracy issues such as: instrument error, unit errors (recording and displaying different systems of units), signal errors, possible unmetered flow, and flow reversal issues. Adjustment factors were applied where possible to correct for unit conversions and signal errors.

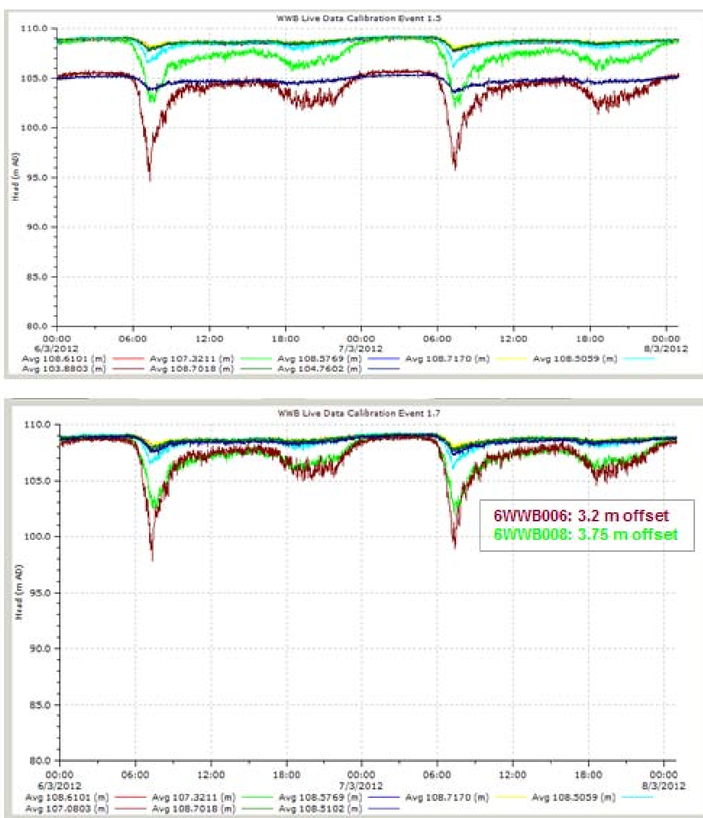

Figure 4. Pressure Gauge Data Validation. The top graph illustrates the original gauged data, with HGL discrepancy during low demand periods. The bottom graph illustrates the corrected data.

CALIBRATION ADJUSTMENTS

The updated model was analyzed under the calibration day conditions to initially compare the model performance against the field data. Upon review of the model performance using standard demand patterns, adjustments would be incorporated to achieve the calibration criteria. Demand patterns typically required adjustment for each pressure zone or discrete metered area (DMA) to calibrate the flows that occurred on the calibration day. An example of calibrated demand pattern is illustrated in the following Figure 5.

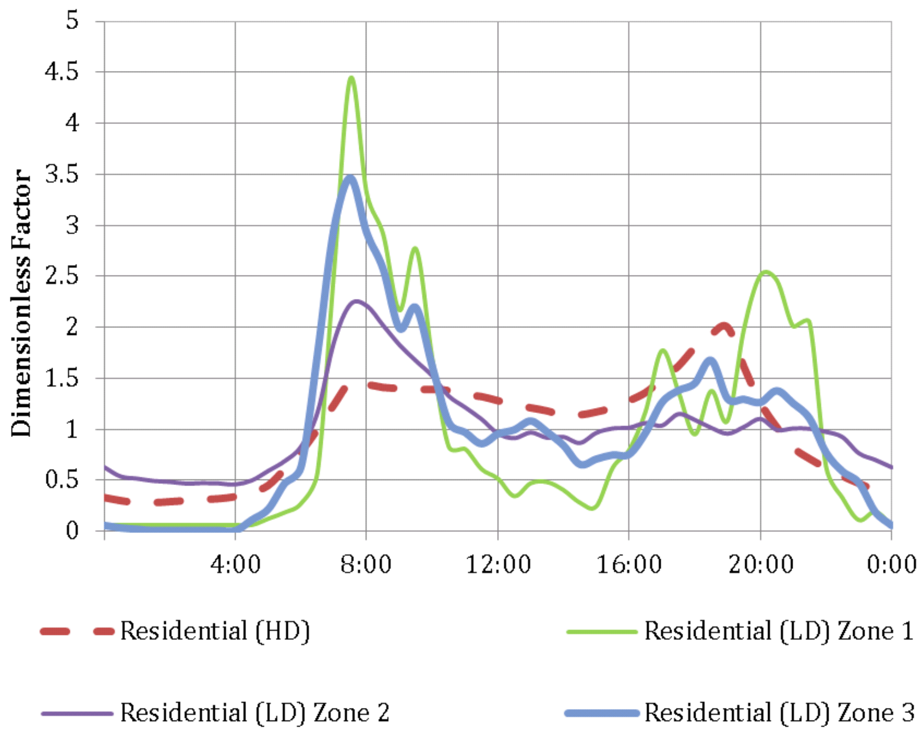

Figure 5. Calibrated Demand Patterns.

Often, pump performance was found to differ from the factory pump curve. The suction pressure and discharge pressure gauges were used in combination with the flow data to calibrate the pump curve to match existing performance. An example graph of an adjusted pump curve is shown in the following Figure 6.

Once the flows at all meters were calibrated by the items above, the model was further evaluated to determine whether the headloss differed between the model and the observed data. Further adjustment would then be necessary, such as adjustment of pipe roughness coefficients.

Adjustments were also performed necessary at key assets, such as valves and pumps to ensure that operation was consistent with that recorded for the calibration day. For example, valve settings were adjusted to partially or fully closed if measured headloss exceeded model predictions. Areas which required significant modifications in order to achieve calibrated pressures, would then be noted for further investigation by the utility.

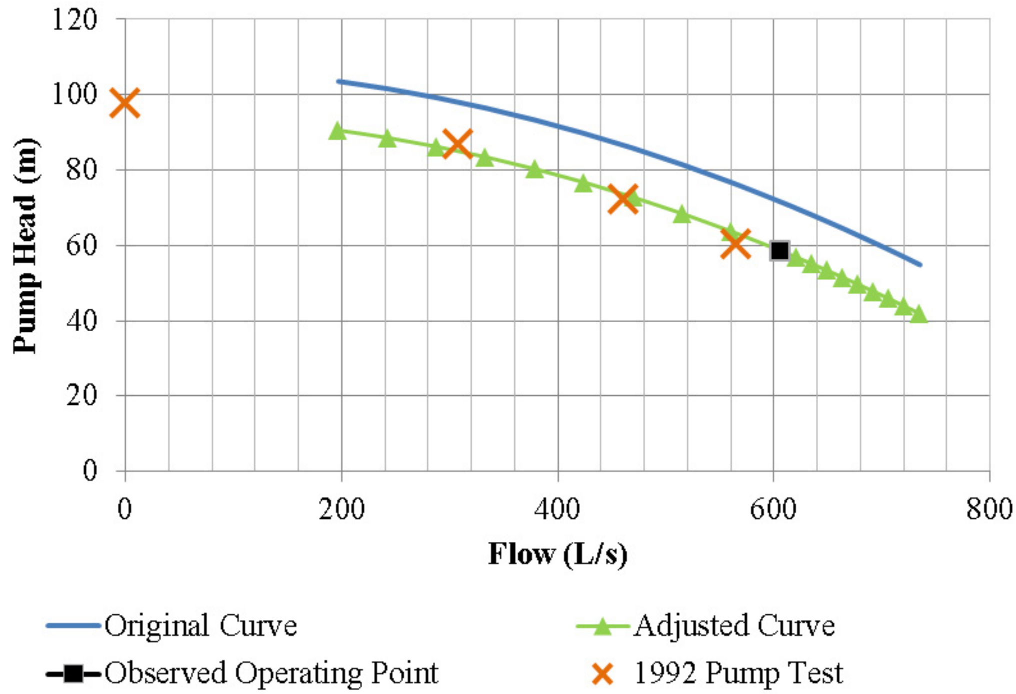

Figure 6. Pump Curve Calibration.

RESULTS

At the completion of the project, 48 distribution models were calibrated, as well as all 11 transmission system models. The calibration tolerance of +/- 1 m difference between observed and measured hydraulic grade over a 24-hour period was achieved for the majority of calibration points. Reservoir levels were calibrated to within 0.5 m of measured data. In order to achieve these criteria in the model the boundary conditions and initial settings were updated based on measured data, the control settings were updated, and the demands were updated to match the demand patterns observed on the calibration day. In some cases additional modifications were needed to achieve the calibration tolerance, such as: pump curve adjustment, valve diameter adjustment, pipe roughness coefficient adjustment, and valve operational status. These additional modifications provided useful information to the utility, such as determining booster pump stations with reduced pump performance. Representative calibration results are shown in Figures 7 through 10.

Upon successful calibration, the models were analyzed for max day, future max day, and fire flow conditions and reviewed for pressure and velocity concerns.

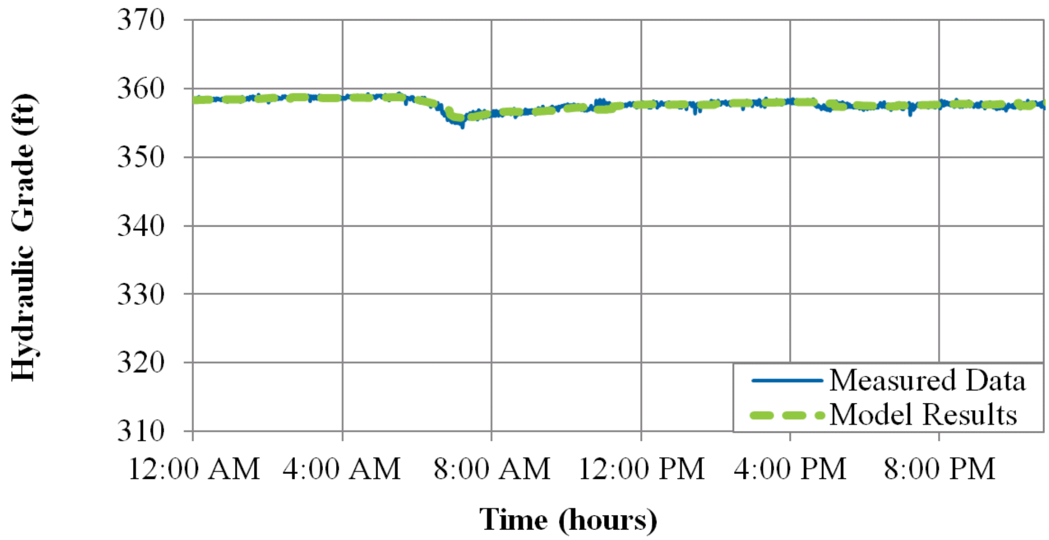

Figure 7. Pressure Gauge Calibration Graph.

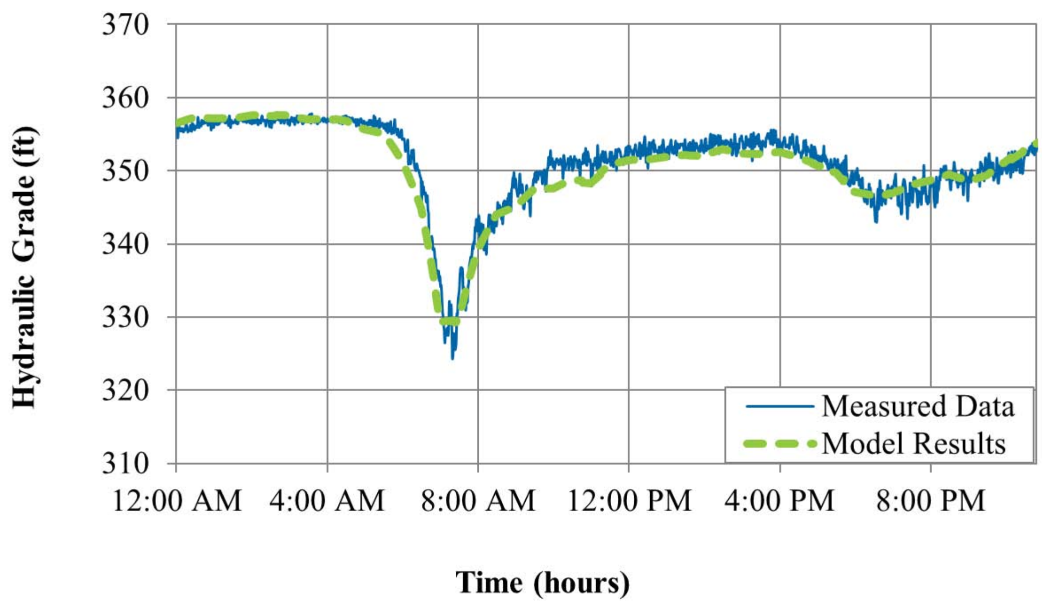

Figure 8. Pressure Gauge Calibration Graph.

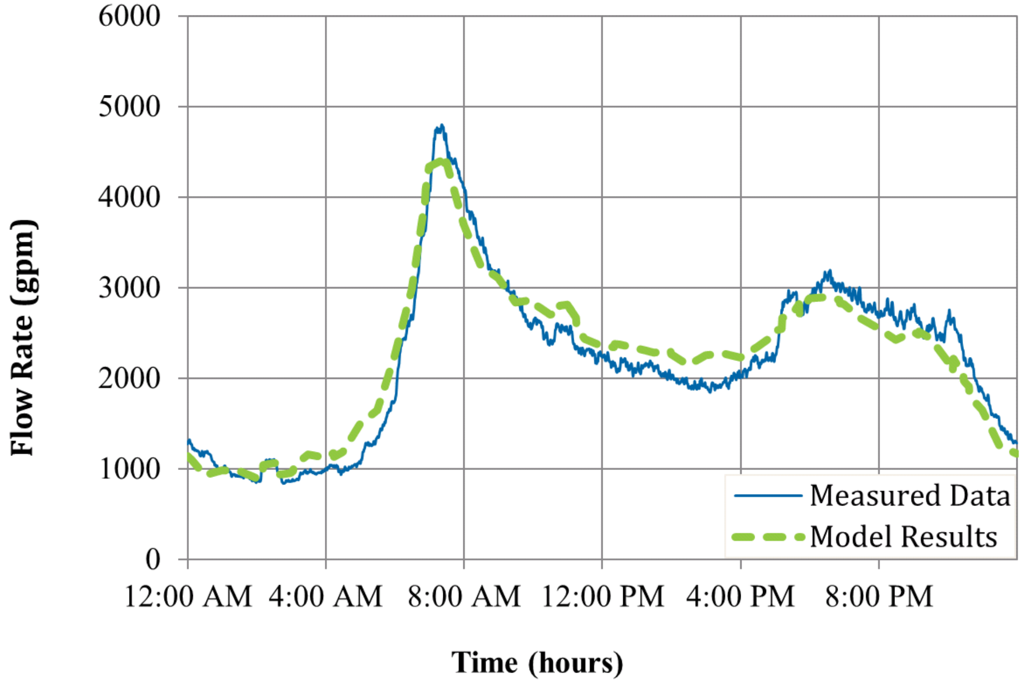

Figure 9. Flow Meter Calibration Graph.

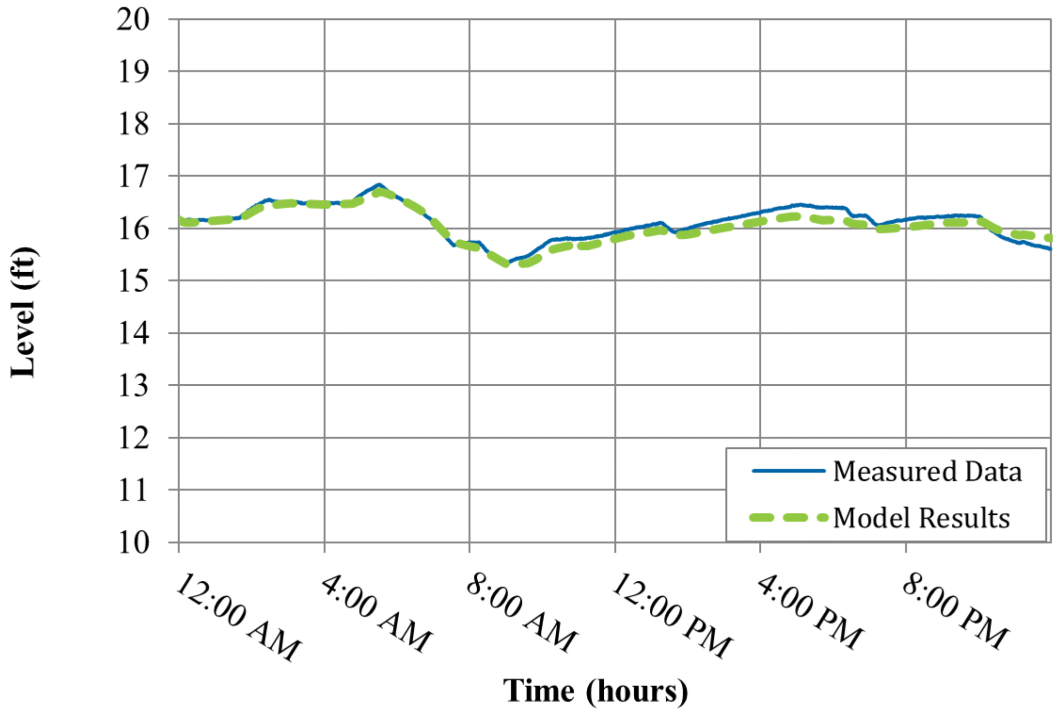

Figure 10. Storage Tank Calibration Graph.

CALIBRATION BENEFITS

In addition to achieving the project goals, the calibration process was able to identify previously unknown issues within the transmission and distribution systems, such as:

- Potential operational issues, such as closed valves or blockages
- Reduced performance, especially for pumps
- Open valves on pressure zone boundaries
- Incorrect GIS data
- Accuracy issues of permanent flow meters and pressure gauges

Identifying these potential issues allowed the utility to focus field efforts on particular portions of the system. Ultimately, Sydney Water obtained models suitable for their needs. The extended period calibration improved the models' ability to predict daily trends, performance of key assets. This allowed the models to serve as a basis for many types of analysis performed by the utility, including anything from reliability analysis, to optimization of controls and energy efficiency.

Analyzing Pump Energy through Hydraulic Modeling

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Abstract

Over the last decade, United Water Toms River (UWTR) implemented an aggressive capital improvement program to incorporate several new facilities into the water distribution system. With the new assets in place, UWTR initiated a pump energy usage analysis to optimize the water distribution system operations. The primary objective of this study was to conserve energy by improving the distribution system pumping efficiency. Using a hydraulic model database, service area boundaries were developed to automate water distribution system operations, improve pumping system efficiency, and maintain network connectivity. New control valves (check, flow control, pressure reducing, and pressure sustaining) and isolation valve closures were identified to delineate the boundaries. The new boundaries provided the framework necessary to update the water system operations for the production facilities and the booster pump stations to reduce the energy required to maintain a uniform level of service. As a result of our hydraulic modeling efforts, the overall system delivery costs decreased by seven percent. The conclusions associated with this analysis are based on model simulation results. UWTR is currently designing the infrastructure necessary to integrate the proposed service area boundaries. Following the service area boundary integration, UWTR plans to implement the recommended water system operations. The project team will be positioned to compare actual field data to the model simulation results following the operational adjustments.

INTRODUCTION

There were two primary objectives of this project: improve water distribution system operations and reduce pumping energy costs. A comprehensive hydraulic model was used to simulate various scenarios, which allowed for a more flexible approach to managing the water system. The hydraulic model was used to evaluate water distribution operational changes and identify inefficient pumping operations.

The project was conducted using a three-part execution strategy. First, service area boundaries were developed, using infrastructure modifications that required minimal capital investment. By establishing local service areas, appropriate pressures could be more efficiently maintained, and tank turnover could be facilitated. Next, water system operations were automated using the infrastructure within each service area; such as pump operations based on tank water levels. Finally, the energy analysis was executed subsequent to implementing the service area boundaries and the operational modifications.

After labor costs, energy costs are the second highest expense for utilities. Pump operations account for up to 80 percent of the energy used within a water distribution system. As a result, managing pump operations provides an opportunity to conserve energy and improve pumping efficiency.

This paper summarizes the energy plan analysis for the United Water Toms River (UWTR) water distribution system pumping operations. The analysis compares the energy requirements of the current system operations to the proposed system operations. With the exception of the proposed North Dover Booster Pump Station and minor improvements to develop service area boundaries, the existing distribution system infrastructure was used for this analysis. The results reveal less energy use, and consequently, lower operating costs for the proposed system operations. By more efficiently using the existing infrastructure and strategically implementing system improvements, the energy costs were reduced by 7 percent.

SERVICE AREA DEVELOPMENT

The water distribution system hydraulic model database included seven production facilities capable of providing up to 24.8 mgd. Figure 1 illustrates the distribution system infrastructure, which includes three booster pump stations, five storage tanks with a combined capacity of 3.8 million gallons, and over 550 miles of pipe.

Prior to beginning the study, the model was evaluated to confirm the accuracy of the reproducing the actual field conditions. Three years (April 2010 through May 2013) of SCADA operations data were analyzed, including information regarding tank levels, pump flow, and delivery pressure. Then the model results were evaluated based on consistency with known maximum month average day (MMAD) demand conditions and maximum day (MD) demand conditions.

The model was updated to correspond with the SCADA data that included pump controls and valve operations, which were incorporated into the model database. The water demands in the hydraulic model were adjusted globally to 19.5 mgd and 25.0 mgd to represent the MMAD and MD time periods, respectively. The model results were iteratively compared with the SCADA data and refined accordingly.

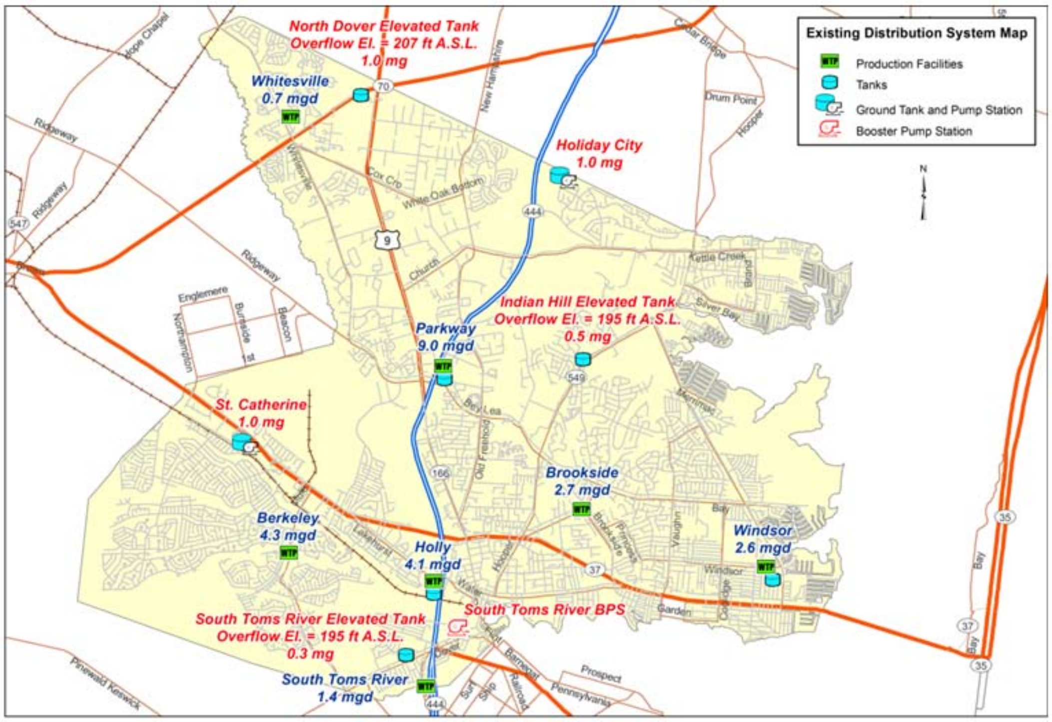

Figure 1. Existing Distribution System Map

Service area boundaries were developed and defined using the hydraulic model database. The current water distribution system operates as a single service area, so all of the production facility operations were affected by the hydraulic grade line (HGL) of the North Dover Tank, which is approximately 12 feet greater than the remaining elevated tanks. Operating the water distribution system as a single service area, the water system was subjected to a wide range in system head conditions, due to the diurnal pattern. The peak factor was approximately 1.8, which resulted in a peak hour demand of 45 mgd, or 20 mgd greater than the maximum plant capacity. As a result, all available water system storage was required to supply the peak hour demands. This is illustrated in Figures 2 and 3, which show the diurnal pattern and the system head curves, respectively. The peak hour was found to be hour 29 in the simulation and the minimum hour was hour 49, as shown in Figure 2.

Three service areas (i.e., Berkeley, Central, and North Dover) were defined to maximize circulation, facilitate redundancy, automate operations, and improve pumping system efficiency. These service areas are illustrated in Figure 4. The use of isolation valves in conjunction with a new booster pump station (BPS), North Dover BPS, were used to define the new service areas, allowing for the water distribution system to maintain a uniform level of service based on the time of year, average demands, maximum demands, and historical peak demand: 10 mgd, 16 mgd, and 25 mgd, respectively.

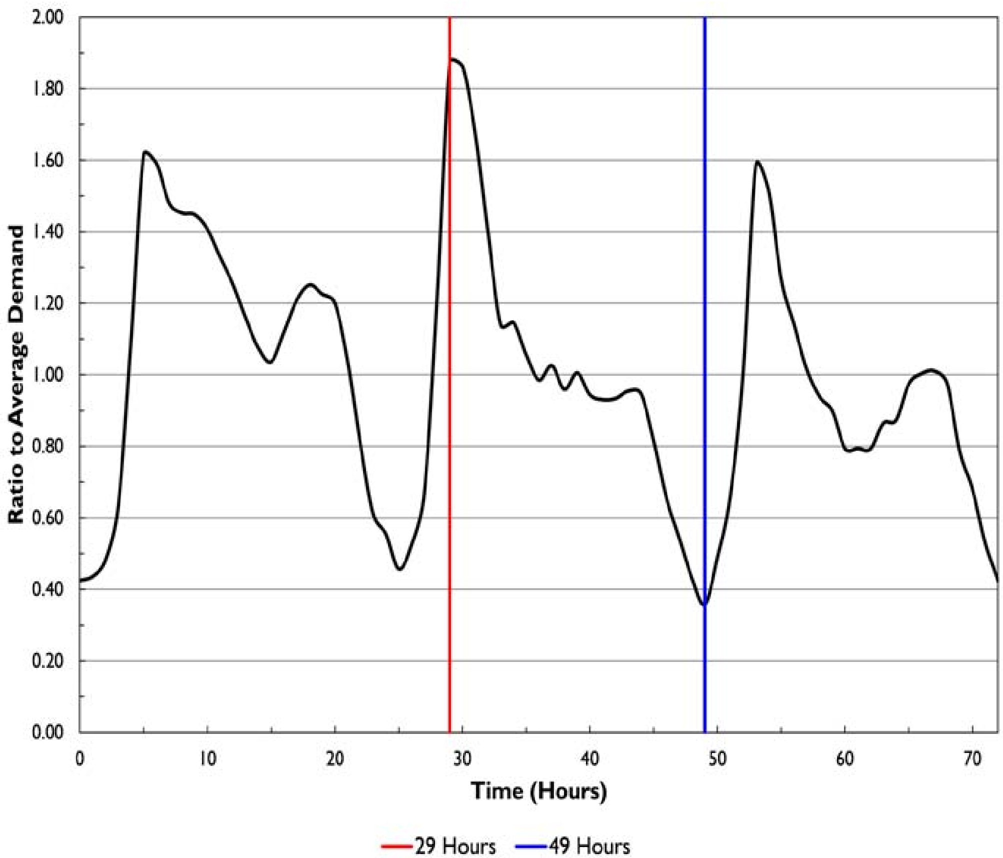

Figure 2. UWTR Distribution System Diurnal Pattern

Using the new service area boundaries, only the Whitesville facility was subjected to the higher North Dover HGL. Also, the Holly facility is partitioned to independently pump west to the Berkeley Service Area and east to the Central Service Area. Both of these changes were intended to reduce energy usage.

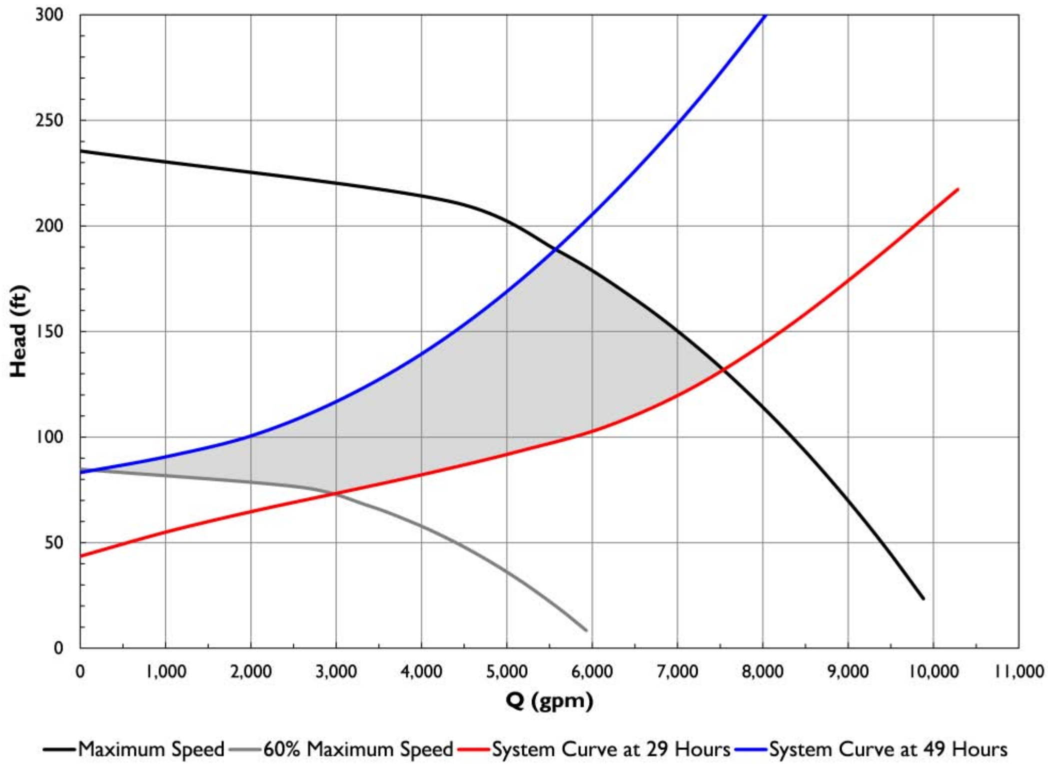

Figure 3. UWTR Distribution System Head Curves

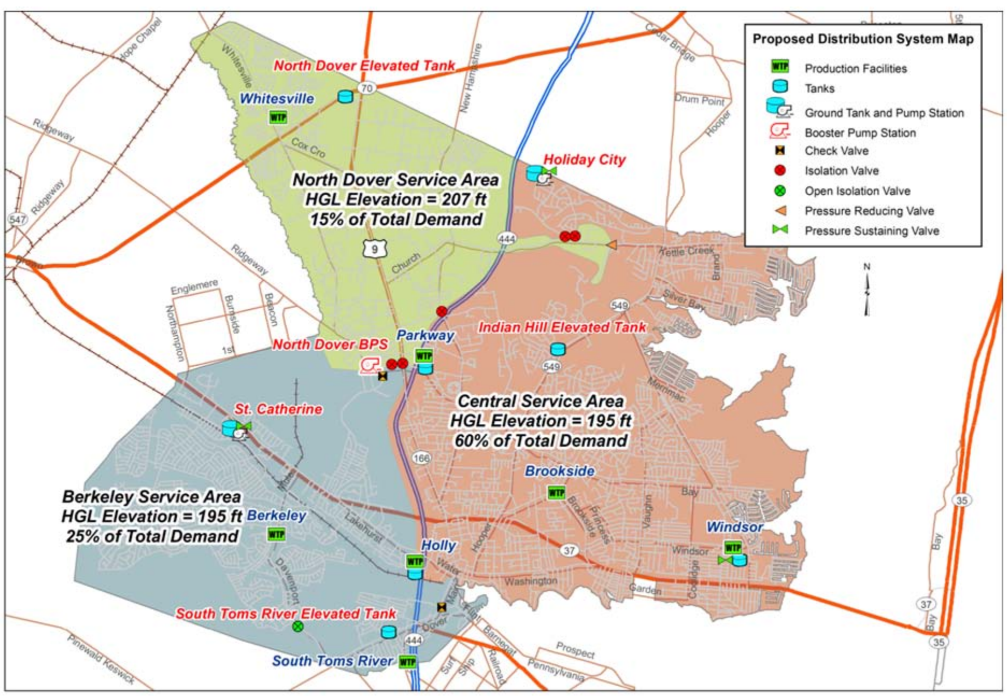

Figure 4. UWTR Proposed Service Areas

ENERGY ANALYSIS METHODOLOGY

The hydraulic model database was used to quantify the energy savings associated with the recommended operational changes. The energy cost module interpolates the pump efficiency and motor efficiency from the pump curve data using the flow and pump head calculated from the model simulation results. Therefore, the pump curves in the model were confirmed and updated, as necessary, to incorporate current operational and efficiency information.

Although pump efficiency was available for each pump in the distribution system, motor efficiency was not reported for all pumps. The pump curve data for Holiday City, Holly, St. Catherine, and Windsor contained information for the wire-to-water efficiency, which is the product of the pump efficiency and motor efficiency. The motor efficiency for the remaining pumps was maintained at a constant value of 92 percent for this analysis. The power (P) and energy (E) associated with each facility were calculated based on the following standard equations 1 and 2:

$$\boldsymbol{P = \frac{\gamma Q H_p}{550 e_p e_m}} \quad [1]$$

$$\boldsymbol{E = P\,t} \quad [2]$$

where γ represents the specific weight of water $\left(\frac{lb}{ft^3}\right)$, Q denotes flow $\left(\frac{ft^3}{s}\right)$, H_p symbolizes the total dynamic head (ft), 550 is the conversion to horsepower $\left(\frac{550\,\frac{ft\,lb}{s}}{1\,HP}\right)$, e_p symbolizes the pump efficiency, and e_m symbolizes the motor efficiency, and (t) is the power delivery time interval.

The module calculates the energy required to operate each pump, which was summarized using a spreadsheet to report the total energy required for the distribution system pumping operations at each facility. The electric tariff was also incorporated into the spreadsheet to calculate the pump operating costs for each facility.

The electric tariff is comprised of three price components, which includes a capacity charge, delivery charge, and generation charge. The capacity charge is based on the monthly peak power supplied, and the delivery and generation charges are proportional the monthly energy consumption. Note that the peak power used to calculate the capacity charge represents the 15-minute maximum power supplied for the monthly billing period.

The rates associated with the delivery and capacity charges vary seasonally (i.e., summer and winter rates), and the generation charge remains constant. The structure of the current electric tariff is summarized in Table 1. The summer rates were used due to the 16 mgd demand condition considered for this analysis. Monthly base charges were not included in the energy cost calculation because these values are negligible relative to the cumulative effects of the electric tariff rates.

Table 1. Current Electric Tariff

Type of Charge	Time of Year	
	Summer	Winter
Capacity	Over 10 kW: Peak Power × \$6.25	Over 10 kW: Peak Power × \$5.83
Delivery	First 1,000 kWh: \$0.065260/kWh	First 1,000 kWh: \$0.061088/kWh
	Over 1,000 kWh: \$0.013900/kWh	Over 1,000 kWh: \$0.013900/kWh
Generation	\$0.08 / kWh	

PROPOSED OPERATIONS

A number of changes in the operations of the water distribution system were proposed to improve the quality of service and reduce water system pump operating costs. These recommendations were made with a focus on two priorities: optimizing operation of the distribution system and minimizing capital spending on infrastructure projects. The water system pumping operations were automated to be controlled based on tank water levels. As a result, the system automatically reacts to demand changes and inherently manages diurnal fluctuations. The service area boundaries provided local zones of influence and allowed the production facilities to work together without competing, which eliminated the need to over-pressurize the southern portion of the system to provide an adequate level in the North Dover Tank. In addition, we recommended restricting the operations at each pumping facility to maintain more efficient conditions.

Several capital improvements were required to implement the water system operational changes. The most significant was the addition of the North Dover Booster Pump Station (BPS). In addition, a number of control valves and strategic valve closures were required to isolate the North Dover Service Area.

ENERGY ANALYSIS RESULTS

The recommended changes in water system operations were applied to the model and those results were compared to known values. Table 2, a 30-day energy analysis summary, shows the results of this comparison for a maximum month average day demand condition (16 mgd). Although energy (kWh) was reduced by 9 percent, the energy cost savings decreased by roughly 7 percent. Because of the electric tariff structure and the addition of the North Dover BPS, energy savings do not translate directly into cost savings. The overall system efficiency was estimated to increase by 3 percent. Note that the overall efficiency estimate received proportional consideration based on energy use (i.e., overall efficiency represents an energy usage weighted average).

Table 2. 30-Day Energy Analysis Summary

Facility	Maximum Power (kW)		Energy (kWh)		Efficiency (%)		Energy Cost ($)	
	Current	*Proposed*	*Current*	*Proposed*	*Current*	*Proposed*	*Current*	*Proposed*
Berkeley	170	190	110,500	104,500	77.4	77.7	11,450	11,000
Brookside	90	105	54,150	50,400	64.9	68.9	5,650	5,350
Holiday City BPS	25	30	3,900	6,200	57.1	68.4	500	750
Holly	125	215	64,500	60,200	76.8	79.4	6,800	7,000
Parkway	360	250	116,850	87,550	69.2	73.3	13,200	9,750
St. Catherine BPS	40	45	7,850	5,750	49.9	44.7	1,000	800
South Toms River	35	80	15,900	28,450	74.2	74.1	1,700	3,150
Whitesville	35	35	22,350	23,450	72.7	72.6	2,300	2,400
Windsor	185	140	50,350	21,050	72.8	67.2	5,850	2,850
North Dover BPS	------	60	------	20,050	------	77.0	------	2,250
Total			446,350	407,550	72.1	74.2	48,500	45,250
Percentage Change			Reduced by 8.7%		Increased by 2.8%		Reduced by 6.7%	

Table 2 reveals that the maximum power used at the Holly facility is predicted to increase, despite the lower energy usage. This can be attributed to the new pumping strategy at that station, in which Holly delivers potable water to the east and west simultaneously. Pump efficiency improved at the Holly and Parkway Pump Stations due to restricting minimum and maximum flow. The pump efficiency at the Windsor facility and the St. Catherine BPS decreased due to the lower head conditions, but these pumps operate less, so the loss in efficiency is outweighed by gains in other parts of the system.

FUTURE COST CONSIDERATIONS

This project focused on decreasing the energy requirements for distribution system pumping based on hydraulic constraints. As newer methods of analysis are becoming more feasible, other parameters can be incorporated to expand the analysis. Techniques such as the use of genetic algorithms to weed out less desirable solutions are becoming more main stream and offer opportunities for future energy analysis. In the future, combining the techniques that were central to the success of this project with newer methods will offer more precision to projects of this type.

CONCLUSION

A hydraulic model of the UWTR water distribution system provided the opportunity to identify and prioritize potential opportunities for improvement within the system. By developing service area boundaries and adjusting pump operations to increase efficiencies, the result was a water system that experienced a reduction in the pump operating costs of 7 percent, a reduction in energy usage of 9 percent, and an increase in efficiency of 3 percent overall.

Benefits of PACP® Version 7.0 Update
NASSCO

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Abstract

The Pipeline Assessment and Certification Program® (PACP ®) is an international standard that has been used for many years for assessment of pipelines, manholes, and laterals. Since 2002, over 20,000 people have been PACP certified. In 2015, NASSCO released a new, improved PACP Version 7.0. The development of Version 7.0 included an unprecedented review by over 100 collections professionals, who collectively, raised the bar on PACP industry standards. The new manual includes technical updates, educational benefits, and a more user friendly format. Technically, there are more detailed explanations of deterioration mechanisms,descriptions of current inspection technologies, and the simplification of Level 1 MACP requirements. Educationally, the manual includes an enhanced color coded chart for Header Codes, as well as a section dedicated to pipe shapes and materials. Finally, a new Asset Management Appendix provides basic guidelines to use PACP to develop Likelihood of Failure, Consequence of Failure, and Risk.

Introduction to NASSCO and PACP

The National Association of Sewer Service Companies (NASSCO) is a nonprofit association that was established in 1976 with the goal of increasing the success of everyone involved in pipeline rehabilitation by the delivery of high quality products through education, technical resources, and industry advocacy. NASSCO's mission is to set industry standards for the rehabilitation and assessment of underground infrastructure and to assure the continued acceptance and growth of trenchless technologies.

The NASSCO Pipeline Assessment Certification Program® (PACP®) was established in 2002 to provide standardization and consistency to the way we evaluate our underground infrastructure. At that time, there was no standardized protocol in the United States for the collection and management of data collected from internal inspection of pipelines. Collection system owners either created their own individual systems, or simply allowed each operator to collect data using no standard at all.

This lack of standards severely limited the value of observations collected by Closed Circuit Television (CCTV) cameras, which have been in use since the early 1960s. Inconsistent observations made it impossible to compare the condition of one pipe to another, even within the same network. Standardization not only made it more practical to compare conditions of multiple segments within a pipe network, but it also allowed the industry to benchmark these conditions to give us a better understanding of the deterioration mechanisms affecting underground infrastructure.

The Pipeline Assessment and Certification Program (PACP ®) is an international standard that has been used for many years for assessment of pipelines, manholes, and laterals. Since 2002, over 14,000 people have been PACP certified. In 2015, NASSCO will release a new, improved PACP Version 7.0.

The development of Version 7.0 included an unprecedented review by over 60 collections professionals, who collectively, raised the bar on PACP industry standards.

The new manual includes technical updates, educational benefits, and a more user friendly format.

Technically, the manual will provide detailed explanations of deterioration mechanisms and descriptions of current inspection technologies.

Deterioration Mechanisms

Factors that influence deterioration of pipelines can be categorized as Structural Related, Maintenance Related, and Construction/Design related. This information should be combined with the internal inspection record in assessing the condition of the pipe.

Structural Related:

- **Soil Quality -** A buried pipe's structural performance is related to the quality of the soils surrounding the pipe.
- **Position of Groundwater Table -** When the pipe is deeper than the groundwater table, it is subject to groundwater infiltration through structural defects.
- **Loads -** The external loads on a buried pipe come from the weight of the soil, the groundwater, and any loads on the ground surface, such as, roadways, railways, and runways.
- **Original Pipe Strength and Its Loss over Time -** Changes in the strength of the pipe structure over time are influenced by its operational environment and the pipe's response to the external loading condition.
- **Alignment and Sags -** Unintended changes in the horizontal and vertical alignment and sags are indicative of soil movement and/or consolidation.

- **Mortar Loss/Bricks Missing in Walls of Pipe -** The number of brick courses or layers is related to the size and the depth of the pipeline, and the size of the bricks. Loss of mortar is to be expected over time, and can cause bricks to fall out or deformation from the original shape of the pipeline.

Maintenance Related:

- **Cleaning Methods -** There is a host of cleaning methods for maintaining flow conditions in the pipe.
- **Roots** - Roots enter through joints, fractures, and other openings in the pipe such as break-in service connections.
- **Fats, Oils and Grease (FOG)** - FOG attaches to the pipe at the flow-line and builds up, decreasing the cross-sectional area, and restricting flow in the pipe.
- **Obstructions/Blockages** - In addition to the pipe's own debris, many foreign objects are found by crews maintaining piping systems; from automobile and truck wheels to tools left behind by others.
- **Improper Pipe Repairs** - Using the wrong pipe size, failing to properly join the repair section with the existing pipe, and/or failing to provide a quality embedment for a pipe repair, can lead to root intrusion, debris collection, and/or flow restrictions.
- **Poor Access to Manholes for Maintenance** - Manholes that cannot be easily accessed increase the level of effort required to provide preventive or reactive maintenance to pipeline.
- **Hydrogen Sulfide (H_2S) Attack or Other Chemical Attack** - The lack of aeration that occurs in force mains and slow moving flows can promote the growth of bacterium that convert gases given off by the fluid into harmful acids or other chemicals.

Construction/Design Related:

- **Surcharging** - Flow being carried by the pipe exceeds the capacity, and the water level (or hydraulic grade line) is above the crown (top) of the pipe.
- **Quality of Construction** - Poor installation techniques and materials can accelerate deterioration of the pipeline.
- **Defective Lateral Connection Methods and Other Defective Junctions** - Poor quality of the service connections such as break-in taps may lead to mainline piping issues such as structural defects, root intrusion, blockages, flow restrictions and surcharges.

Supplemental Technologies

While CCTV is the primary means of pipeline assessment, other technologies have been developed. Some of these technologies are capable of providing a higher level of accuracy than a CCTV technician can provide visually using a 2-dimensional view. Many of these technologies can be used in conjunction with CCTV, such as Laser Profiling, or Sonar.

The following are short descriptions of some of those tools available. As these tools become accepted in the industry, the quantifiable information ranges will be adjusted accordingly to represent the ever-changing state of the art.

- **Laser Profiling** - The function of the laser profiler is to provide the engineer with accurate data of the existing shape and or condition of the internal wall of the pipe being inspected. These systems are typically utilized to report pipeline deflection, deformation, ovality, and changes in cross sectional area. Laser profilers may also be used to estimate wall deterioration (loss of thickness of the pipe wall). The digital profile data can be used to report the data collected in several useful formats, such as: graphical data, 3D image, fold flat image.
- **Laser Diode Measurement Tools -** Laser diode measurement technology is utilized by pipeline inspectors to measure defects inside the pipe during normal CCTV inspections. Typically this type of measurement tool is used for crack and fracture measurement, joint openings, and other issues of concern that need measurement to provide proper perspective and accurate data to the engineer in order to establish benchmarks for existing pipe and acceptability of new pipe.
- **Sonar -** The sonar profiler is designed to provide dimensional data on debris levels, grease accumulation, pipe deformation and other anomalies below water level where visual inspection cannot be used. In surcharged lines or siphons the sonar can provide the profile and dimensional data of significant obstacles or defects.
- **Sidewall Scanning -** Panoramic view inspection systems are digital imaging cameras that are capable of a continuous 360 degree image capture of the wall of the pipeline being inspected. Due to the high definition of the digital image, quality the inspections may be conducted at a higher speed than traditional CCTV methods.
- **Zoom Camera Technology -** The primary function of the zoom camera inspection is to obtain a preliminary diagnosis of the pipe segment by observing its condition entirely from the access point. This technology consists of a telescopic boom for lowering the camera into the access point, and a high-powered zoom camera.
- **Pipe Penetrating Radar (PPR)** - This technology works similar to "ground penetrating radar" in that electromagnetic waves are sent through the pipe wall from within the pipe to identify exterior voids, approximate wall thickness (+/- 10%), presence of reinforcement steel, exterior repair couplings, and changes in soil or water content to a distance of three feet.

Color Coded Charts' Enhancements

Educationally, the manual includes an enhanced color coded chart (see Figure 1) for Header Codes which facilitates completion of each field.

Section 2 — Header Form Fields

20 Sewer Use 2-8
SS = Sanitary
SW = Stormwater
PR = Processes
CB = Combined
FM = Force Main
XX = Not Known
ZZ = Other

21 Direction 2-9
U = Upstream
D = Downstream

22 Flow Control 2-9
P = Plugged
L = Lift Station
B = Bypassed
N = Not Controlled
D = Dewatered Using Jetter

25 Shape 2-10 D-1
A = Arched
B = Barrel
C = Circular
E = Egg-shaped
H = Horseshoe
O = Oval (elliptical)

25 Shape 2-10 D-1
R = Rectangular
S = Square
T = Trapezoidal
U = U-Shaped with Flat Top
Z = Other

26 Material 2-10 D-4
AC = Asbestos Cement
ABS = Acrylonitrile Butadiene Styrene
BR = Brick
CAS = Cast Iron
CMP = Corrugated Metal Pipe
CP = Concrete Pipe

26 Material 2-10 D-4
CSB = Conc. Segments Bolted
CSU = Conc. Segments Unbolted
CT = Clay Tile
DIP = Ductile Iron Pipe
FRP = Fiberglass Reinforced Pipe

26 Material 2-10 D-4
OB = Orangeburg/Pitch Fiber
PCCP = Pre-Stressed Concrete Cylinder Pipe
PCP = Polymer Concrete Pipe
PE = Polyethylene
PP = Polypropylene

26 Material 2-10 D-4
PSC = Plastic/Steel Composite
PVC = Polyvinyl Chloride
RCP = Reinf. Concrete Pipe
RMP = Reinf. Plastic Pipe
SP = Steel Pipe
SB = Segmented Block

26 Material 2-10 D-4
VCP = Vitrified Clay Pipe
WD = Wood
XXX = Not Known
ZZZ = Other

27 Lining Method 2-11 D-17
CP = Cured-In-Place Pipe
FF = Fold and Form
GRC = Glass Reinf. Cement
SW = Spiral-Wound
SC = Continuous Slip Liner
SE = Sectional Slip Liner
SN = Segmented Panel

27 Lining Method 2-11 D-17
SP = Segmented Pipe
GP = Grout-In-Place Liner
FP = Formed-In-Place Liner
SL = Spray Liner
XX = Not Known
ZZ = Other

27a Coating Method 2-12 D-23
EP = Epoxy
PO = Polyurethane
PU = Polyurea
CT = Coal Tar
CM = Cement Mortar
XX = Not Known
ZZ = Other

34 Purpose 2-14
A = Maintenance
B = Infiltration/Inflow Invest.
C = Post-Rehabilitation
D = Pre-Rehabilitation
E = Pre-Acceptance
F = Routine Assessment

34 Purpose 2-14
G = Capital Improvement Program Assessment
H = Resurvey
R = Pre-Existing Video
X = Not Known

36 Pre-Cleaning 2-15
J = Jetting
H = Heavy Cleaning
N = No Pre-Cleaning
X = Not Known

Figure 1. Enhanced Color Coded Chart for Header Codes

Manhole Diagram

A manhole diagram (see Figure 2) was added to facilitate in understanding and taking manhole measurements and associates them with MACP Fields.

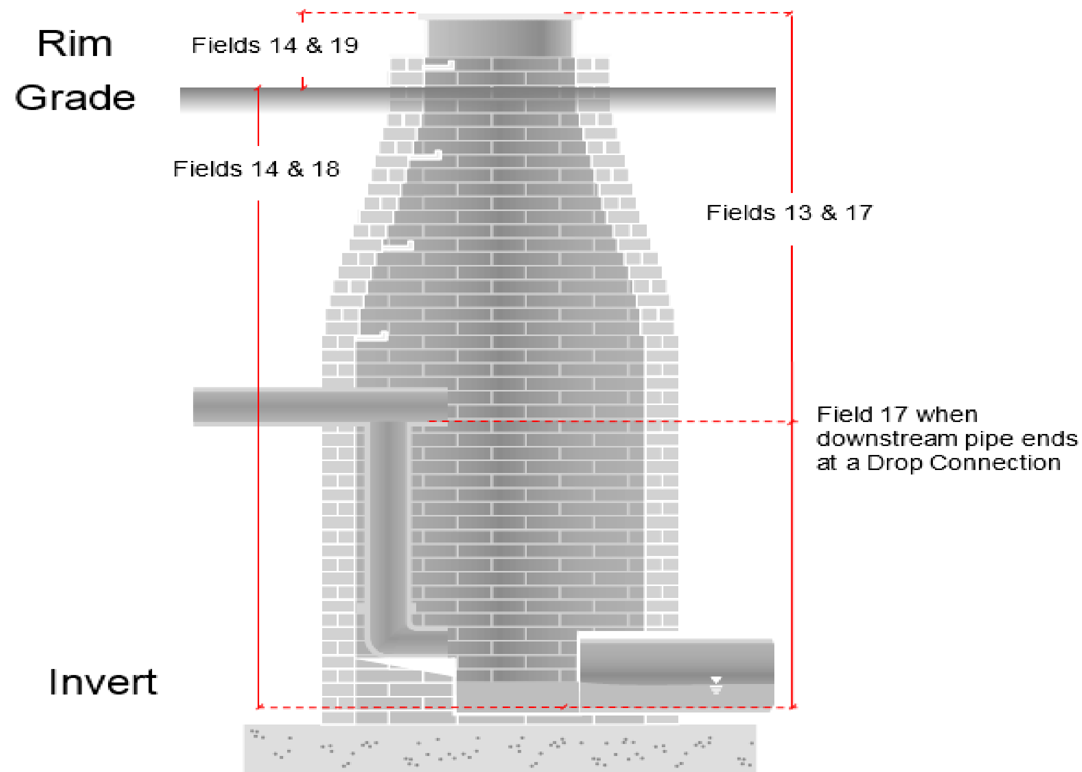

Figure 2. Manhole Diagram

<u>**New Appendices**</u>

Appendix E – Shapes and Materials

In addition to the enhanced color-coded chart, educationally, a new Appendix was added dedicated to pipe shapes and materials. Several examples (see Figures 3 and 4) follow:

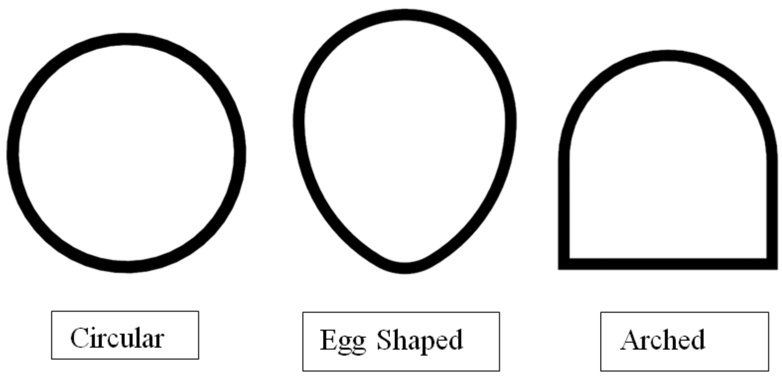

Figure 3. Pipe Shapes

Polyvinyl Chloride (PVC)

Smooth walled plastic generally green or white.

Asbestos Cement (AC)

Can look similar to concrete pipe (CP). However the primary difference is in joint length. AC pipe is comprised of asbestos fibers and cement, while concrete pipe is comprised of aggregate and cement.

Figure 4. Pipe Materials

Appendix D – PACP Based Risk Management

A new PACP Based Risk Management Appendix defines risk management, and then provides calculations to convert the PACP quick rating to a likelihood of failure (LoF) score. It then provides sample calculations and assumptions used to calculate Consequence of Failure (CoF) separately from LoF. Risk is determined by plotting CoF and LoF together (see Figure 5), and recommendations can be based on specific aspects of the risk calculations.

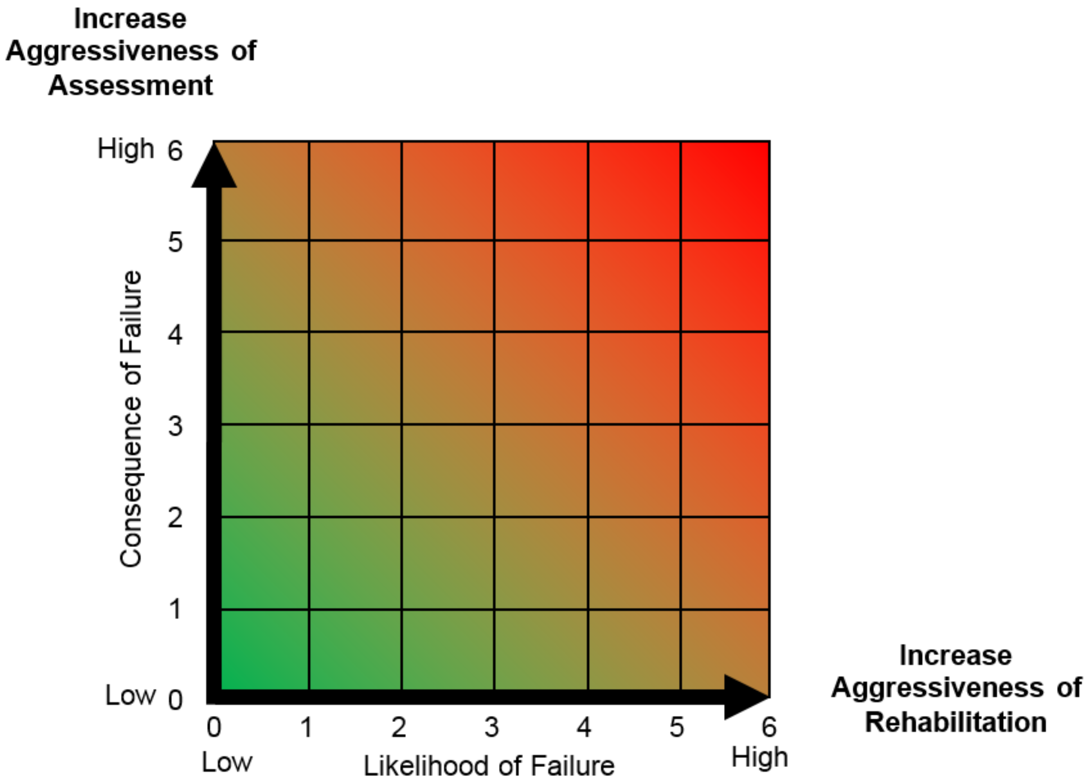

Figure 5. PACP Risk Based Management Graph

Summary

PACP's impact in the industry is huge and will continue to increase as agencies appreciate the importance of standardization and thereby eliminating objectivity. Collecting sound information on infrastructure using established standards provides data to successfully manage an asset well into the future.

As more agencies, such as the Corps of Engineers for Levee pipes and DOT for storm water pipes, assess their pipes, they are also finding the benefits of having a standard. With new uses come new needs, and NASSCO has been working with these agencies to meet these needs while maintaining PACP's strengths that have made it a standard.

In NASSCO's recently published "Pipe Condition Assessment Using CCTV" and "Sewer Pipe Cleaning" Performance Specification Guidelines, a PACP Header Field Checklist was added requiring non-mandatory fields be populated. The specification writer will be responsible for determining which additional fields, beyond mandatory ones, are required for a particular project.

The Condition Assessment of a 30-inch Ductile Iron Water Line by WaterOne of Johnson County, Kansas

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Abstract

WaterOne of Johnson County, Kansas provides potable water to over 140,000 service connections with a production capacity of 200 million gallons per day. A 30-inch unwrapped ductile iron highly critical transmission main was nearing its 50 year life expectancy for this very corrosive soil area. Before replacing the entire 30-inch water main in 2014, WaterOne decided to perform a condition assessment of this pipeline to determine if the full replacement could be delayed for a significant savings to the utility. Remote Field Technology was utilized to determine the corrosion locations and the remaining wall thickness on the transmission main. After verification of the inspection data, WaterOne determined 10% of the pipeline should be replaced within a year, 10% more in five years, and the remaining 80% in 2025. This staggered delay of full replacement led to a $1.8 million total savings based on the time value of money.

INTRODUCTION

WaterOne of Johnson County, Kansas provides drinking water to a population of 410,000 for 17 municipalities on the Kansas side of the metropolitan Kansas City area. The service territory covers 272 square miles with 140,000 service connections. WaterOne has a water production capacity of 200 million gallons per day and approximately 2,600 miles of pipe; 2,400 miles of 16-inch and smaller mains (distribution mains) and 200 miles of 20-inch and larger mains (transmission mains). WaterOne believed a five mile long, 50 year old, 30-inch unwrapped ductile iron (DI) water main would benefit from a major condition assessment project to determine if the life of the pipe could be extended by making repairs or selective replacement of only the most corroded sections in lieu of a complete pipeline replacement. Based on the life expectancy for unwrapped DI in this highly corrosive soil area, its break history, criticality of the water main, and high consequence of failure, this pipeline was scheduled for replacement in 2014 at an estimated replacement cost of $10 million. It was determined if WaterOne could delay full replacement by 10 years,

$2.7 million could be saved; therefore, the inspection process was implemented. In January 2014, WaterOne entered into a contract with Pipeline Inspection and Condition Analysis, Corporation (PICA) to inspect and determine the remaining wall thickness using the electromagnetic Remote Field Technology.

BACKGROUND OF THE PIPELINE TO BE INSPECTED

There is a history to this highly critical water main. The five miles of 30-inch DI transmission main (TM) were installed in 1964. It crosses two major highways, a railroad, and a creek, runs along a roadway, goes through a neighborhood within close proximity to homes, and terminates at a pumping station. In addition, the pipe is one of two TMs that supplies water to two separate pump stations and reservoirs (PS&R) that re-pumps the water to two pressure zones known as the Woodson System and the Northeast RPA (see Figure 1). These two pressure zones consist of approximately 73,000 people or 31,000 service connections. When the 30-inch DI pipe fails, one of the two PS&Rs is no longer available to supply water to the two pressure zones. When both lines are operational, the supply capacity of the two PS&Rs is more than adequate to meet the demands during maximum load conditions. This is not the case when the 30-inch pipe is out of service. The supply capability to the area would be totally reliant on one pipe so a failure on the second pipe or any equipment in the remaining PS&R becomes crucial.

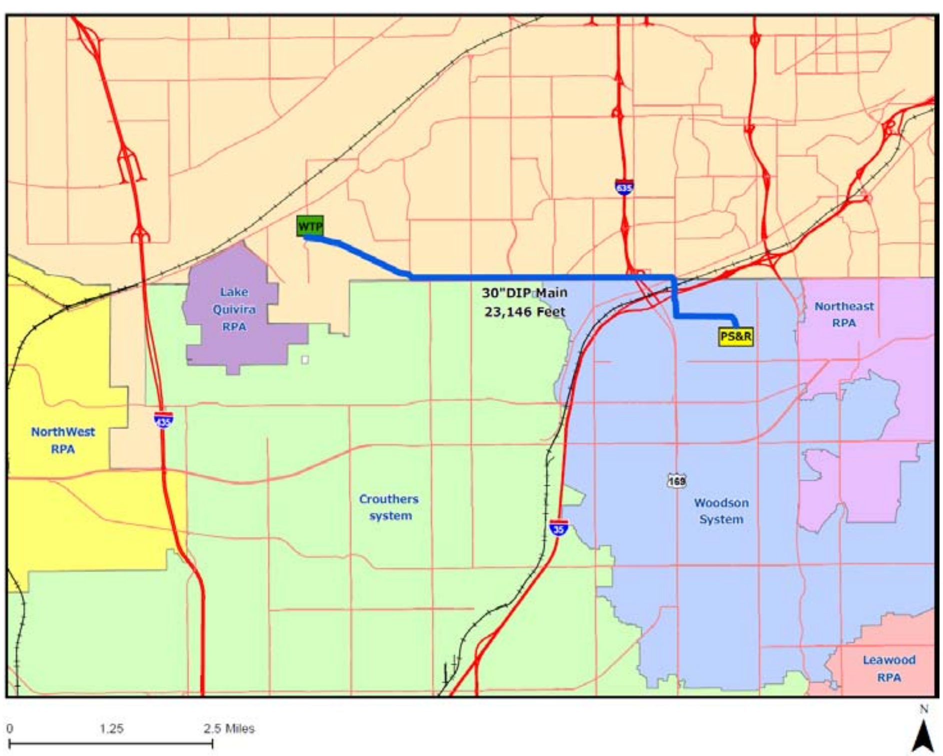

Figure 1. 30-inch Ductile Iron Transmission Main Location

Approximately one of the five miles of the 30-inch TM has already been replaced due to a city street improvement project, leaks, and a hillside erosion problem. The failure mode of this pipeline has been due to localized corrosion resulting in a blowout of the pipe wall leaving a hole the size of a baseball. This pipeline has experienced three breaks in the remaining four miles. One of the failures occurred in June, during the height of load season, resulting in water restrictions to the two pressure zones it feeds. With this information, a full pipeline replacement had been scheduled, but a condition assessment (CA) needed to be evaluated first.

ANALYZING THE CONDITION ASSESSMENT TECHNOLOGIES

WaterOne issued a Request For Proposal in June of 2013 and received proposals from two companies for the CA of the cement mortar lined 30-inch TM. Pipeline Inspection and Condition Analysis, Corporation (PICA) submitted two quotes: 1) free swimming See Snake and 2) tethered See Snake. The See Snake tool uses electromagnetic Remote Field Technology (RFT) to determine the remaining wall thickness (RWT) of the metallic pipe. Pure Technologies submitted quotes for using 2 different technologies: 1) Magnetic Flux Leakage (MFL), an electromagnetic technology, to determine the RWT and 2) PipeDiver technology to determine the average wall thickness (AWT) of each pipe segment. After analyzing all four methods/technologies, it was determined the PICA tethered See Snake tool would be best suited for the inspection process. Although the PipeDiver was the lowest quote received, WaterOne felt the AWT was not precise enough for the analysis needed for evaluating the delay of the replacement of the 30-inch TM. A high resolution data set of the entire circumference of the pipe wall was needed for the comprehensive structural evaluation of the pipe. As a result, PICA was chosen for the CA of the 30-inch main utilizing RFT since they were the lowest quote received for the RWT data. An illustration of how the PICA tool works is shown in Figure 2.

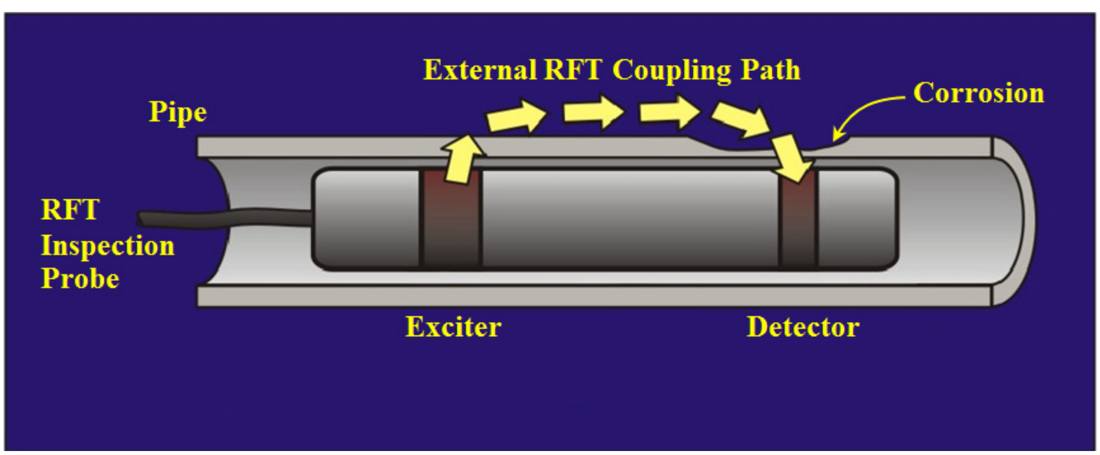

Figure 2. Schematic of Electromagnetic Interaction between RFT Tool and Pipe

EVALUATING THE INSPECTION METHODS

The inspection processes, a tethered versus a free swimming See Snake, were evaluated based on the tool requirements dimensionally, the options of propulsion, the preparation work, and costs. The RFT tool is a full diameter device requiring tight tolerances in relation to the pipe wall to achieve accuracy. For the free swimming approach, its length required a considerable launch point apparatus for a full tool insertion under pressure. The propulsion in the free swimming method would be recommended in most cases, for maneuverability reasons, allowing the water flow to carry the tool from the launch point to the extraction site. This process required a flow of approximately four feet per second. In addition, the pipeline would require pigging for the free swimming method to verify that it was unobstructed and the tool could pass safely from the launch site to the extraction point. The pigging process would produce an excessive volume of water that would not be able to pass into the distribution system. It would be problematic to discard the 4 to 5 million gallons of water at the locations needed for extraction.

The other option of propulsion was a tethered winch method pulling the tool through the TM in 3,500 ft. maximum increments with a combined 270 degree bend restriction per section due to the winch line. This process required a tag line be installed prior to the inspection date capable of pulling the winch line through each section to launch the tool. In the tethered approach, the RFT tool could be assembled in sections and inserted into the dewatered TM. Preparation for both methods required the six existing valves be removed prior to the inspection and necessitated "must dig" sites. Two sections of this TM had previously been replaced with high-density polyethylene (HDPE) pipe with a smaller inside diameter than DI pipe and would not accommodate the diameter of the See Snake; therefore, an extraction of the tool prior to these locations would be necessary for both methods and required "must dig" sites. After evaluation of all of these issues, the tethered method was determined to be a more viable and cost effective approach for this project.

PREPARING FOR THE INSPECTION

The preparation work was extensive and required the TM be taken out of service for an extended period of time. A section of the pipe with a 90 degree bend was sent to PICA for sizing and calibration for the See Snake tool. The existing four miles of DI main were divided into 11 sections utilizing "must dig" sites as needed. The sections ranged in length from 330 ft. to 3,420 ft. Two short sections would require manually pushing the tool through the pipeline and extracting it in the reverse direction with the winch. Each section was mapped and overlaid on Google Earth to aide in strategy discussions with PICA, minimizing pre-inspection site visits. A comprehensive schedule of preparation and inspection work was developed with a timetable for main outage and construction progress to track individual costs. All work was coordinated as close to the inspection date as possible to minimize impact to the distribution system.

The original 1960's construction of the main allowed for the butterfly valves (BFV) to be installed in a six ft. by eight ft. vault with an access manway near the valve for disk seat repair. Using the valve spacing as a starting point, each BFV was removed along with old fittings and vault lids. Vaults in the roadway required new structural lids, which were constructed with oversized manhole lids centered and installed on the existing vaults. This allowed the RFT tool to pass through the opening, be assembled inside the vault, and pushed into the open segments of pipe to accommodate its length. This approach minimized the impact to traffic at the time of inspection and rehabilitation. In vaults without traffic impact, the valves and fittings were removed, open ends of pipe secured to prevent contamination, and the excavation was covered and fenced.

The additional sites were selected near the sections of the HDPE pipe replacement. A majority of the pipeline was in developed areas in which minor obstructions had to be handled such as parking lots, retaining walls, trees, and landscaping. Securing the excavations was not difficult and proposed minimal risk to the public. In these locations, a full 16 foot section of pipe was removed to allow the tool to be extracted in one piece, reducing the time needed between runs.

As mentioned previously, if the See Snake was to be launched in a free swimming insertion manner, it would require a pig to verify the pipe was clear of obstructions. In a tethered approach, it was still required to verify the main was clear of debris and unknown obstacles. It was decided to perform a visual inspection of the inside condition of the pipeline at the same time the tag line was inserted to support winch operations. This, in turn, saved WaterOne expenses since the contractor did not have to perform the pigging.

THE INSPECTION PROCESS

The See Snake inspection process required a winch with a special stretch resistant rope to prevent the RFT tool from surging as it was pulled through the pipe. The surge effect would hinder the accuracy of the footage calculations and make it harder to interpret the location of the data. The winch was a hydraulic accessory supplied by PICA and was supported using a skid steer loader with a front mounting attachment.

See Figure 3 for the winch configuration. The winch rope was tied to the tag line and pulled back to the insertion point using a truck. Caution was taken in this process because occasionally the tag line would get hot near the fittings and would break from the friction. It is recommended that a heavier tag line be used because the 1,200 lb. line was problematic in a few locations and had to be manually retrieved in the pipe.

Figure 3. Winch Configuration for Inspection

Once the rope and winch were in place for a section of the TM, the inspection process began. A boom truck was used to lower the See Snake in segments into the manway excavation sites or in one long section at the large excavation sites. (See Figure 4.) The See Snake had three sections: exciter, body, and receiver. The three pieces were assembled and inserted into the pipe. Set up and calibration generally took an hour each time. The winch line was attached to the front of the RFT tool and an additional winch line was attached to the rear of the tool. The rear line served two purposes. First, to provide resistance to the tool and minimize surging as it moved through the pipe. Second, to pull the tool back for a reverse run to verify the first pass findings and gain accuracy. The tool was pulled through the pipe at a rate of 10 to 11 feet per minute, so the time needed for each pass could be considerable based on its length. It was necessary to complete the section once started.

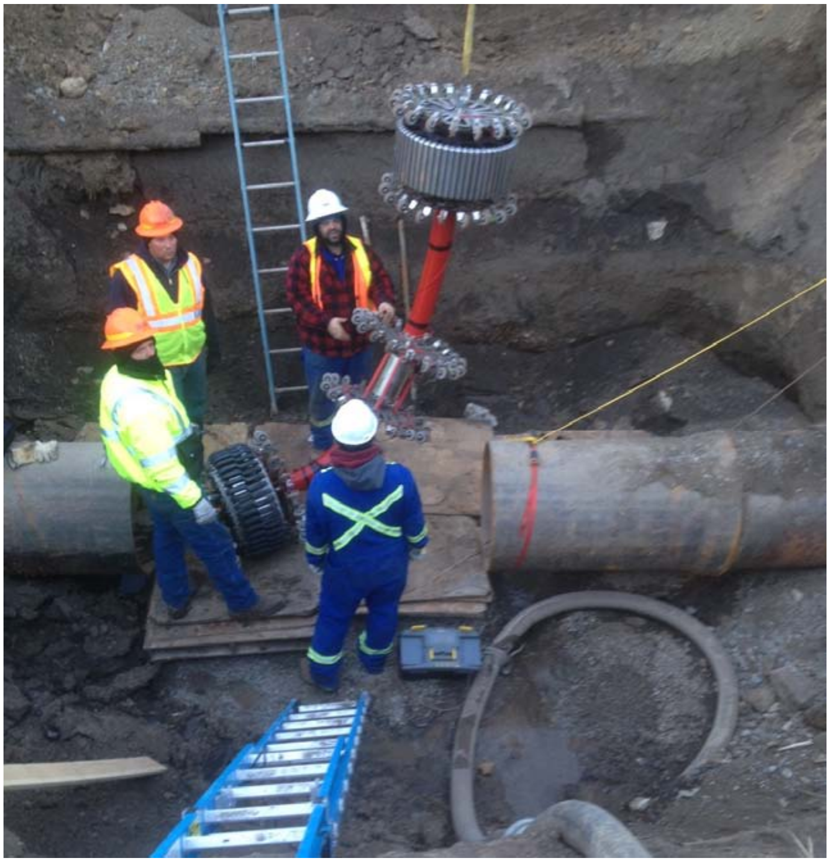

Figure 4. Remote Field Technology Tool for Inspection

The actual inspection went well. The only problem encountered was the unexpected tension put on the See Snake at 90 degree bends. The RFT tool was pulled from the front end. When it was trying to navigate the 90 degree bends, pulling from the front end caused it to pull sideways prior to making the turn. In these cases, it was necessary to guide the tool through the bend manually by pulling or pushing it by hand from inside the pipe. This may not be a problem in a free swimming inspection because there should be no lateral force placed on the See Snake as water would push from the rear; hence the recommendation to perform a free swimming inspection if possible.

ANALYZING THE CONDITION ASSESSMENT RESULTS

The results of the condition assessment utilizing Remote Field Technology were extraordinary. The information received from the inspection showed that the 30-inch transmission main was in fairly poor condition. The RFT was able to provide detailed locations of significant wall loss. Results were provided in a detailed report to WaterOne indicating the three worst wall loss locations in each individual pipe segment for a section of TM in the form of a bar graph (Figure 5), table (Table 1), and diagram (Figure 7). The bar graph is a quick and useful representation of the overall condition of the pipeline because each vertical bar shows the three worst corrosion pits, represented by a diamond, for each pipe segment in that section. The table gives much more detailed information for the three worst wall loss locations. The diagram quickly shows the overall locations of the corrosion spots in relation to both longitudinal and clock position.

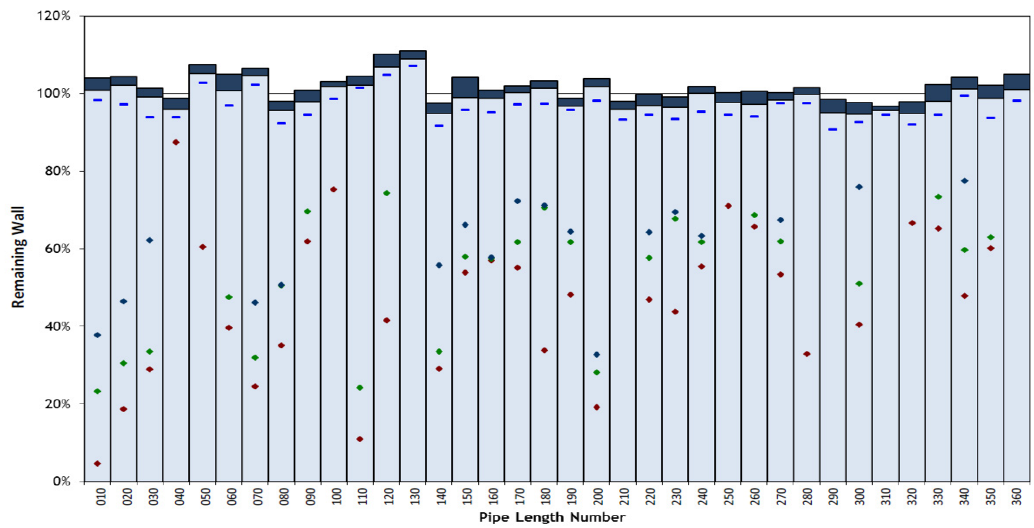

Figure 5. Remaining Wall Thickness per Pipe Section

Table 1. Detailed Pipe Segment Information

Table 5: Wall Thickness Readings – WaterOne 30-in Woodson Supply Main - Section 5

Local Wall Thickness: *Clock positions are with West to East perspective (ie 3:00=South, 9:00=North). Local wall loss entries in red are pitting indications with <15% remaining wall.

Pipe Number	Pipe Location			NWT Pipe Type (inch)	Tavg RW (%)	Circumferential Wall Thickness		Local Wall Thickness: Tmin1			Tmin2			Tmin3			Comments
	Start (ft)	End (ft)	Length (ft)			Tcircmax RW (%)	Tcircmin RW (%)	RW (%)	Location (ft)	Clock Position	RW (%)	Location (ft)	Clock Position	RW (%)	Location (ft)	Clock Position	
0010	1.00	6.50	5.50	0.47													Pipe Type Ia; Partially inspected; First datum point is 1ft from cut-end at Pit 5.
0020	6.50	25.71	19.21	0.47	99%	102%	95%	53%	20.16	1:00	57%	9.51	0:30	76%	11.70	0:30	Pipe Type Ia;
0030	25.71	45.98	20.27	0.47	102%	105%	96%	24%	28.77	1:00	28%	28.22	1:00	80%	40.04	7:00	Pipe Type Ia;
0040	45.98	65.59	19.61	0.47	96%	100%	94%	36%	60.13	0:30	42%	59.38	1:00				Pipe Type Ib;
0050	65.59	85.22	19.63	0.47	99%	102%	96%	44%	75.42	5:00	52%	67.52	1:00	70%	79.89	1:00	Pipe Type Ia;
0060	85.22	104.91	19.69	0.47	94%	98%	92%										Pipe Type Ib;
0070	104.91	124.17	19.26	0.47	101%	104%	98%	72%	106.98	5:00	79%	110.67	6:00	80%	110.10	6:00	Pipe Type Ia;
0080	124.17	144.28	20.11	0.47	97%	99%	92%	60%	127.21	0:30	68%	130.13	6:30				Pipe Type Ib;
0090	144.28	163.11	18.83	0.47	95%	98%	93%	71%	149.08	4:30							Pipe Type Ib;
0100	163.11	183.15	20.04	0.47	96%	99%	91%	17%	165.48	1:30	45%	165.57	12:00	59%	177.62	0:30	Pipe Type Ib;
0110	183.15	202.88	19.73	0.47	99%	103%	95%	27%	197.71	0:30	58%	189.36	6:00				Pipe Type Ia;
0120	202.88	222.95	20.07	0.47	100%	102%	95%	16%	211.73	7:00	41%	209.02	6:30	48%	217.65	0:30	Pipe Type Ia;
0130	222.95	243.15	20.20	0.47	95%	97%	91%	22%	242.37	1:00	70%	237.56	5:00	76%	236.38	5:00	Pipe Type Ib; Dig Sheet
0140	243.15	262.25	19.10	0.47	101%	103%	98%	4%	261.62	0:30	11%	260.23	0:30	20%	256.29	1:00	Pipe Type Ib; Dig Sheet AOIs Sta 107+90 & +91

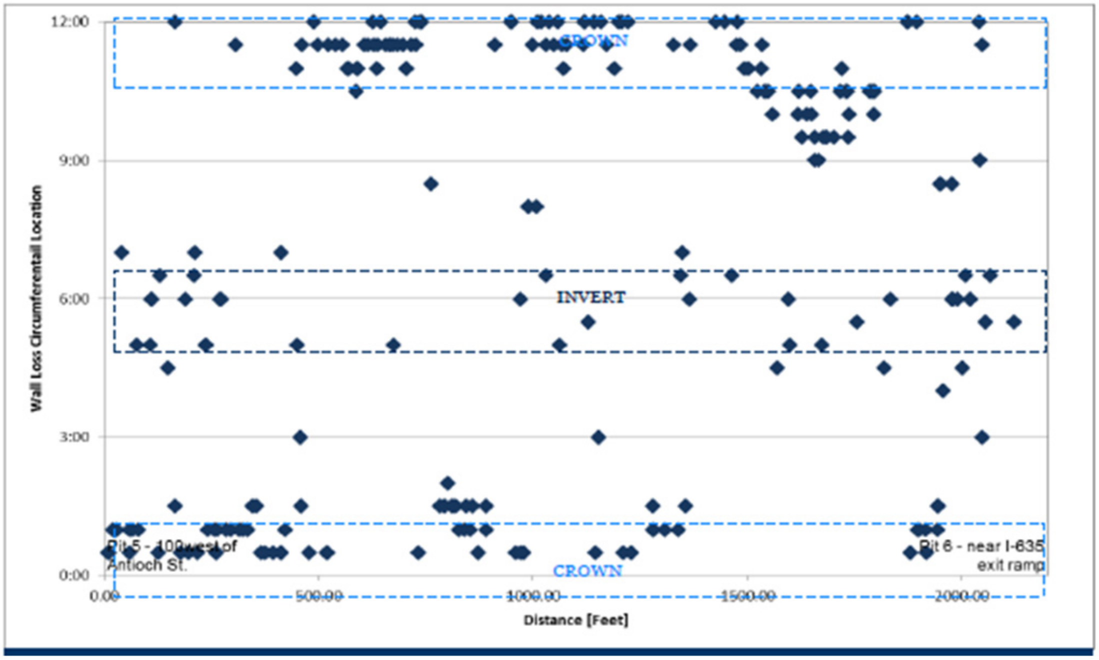

Figure 7. Circumferential Distribution of Corrosion

In order to verify the data PICA provided was accurate, WaterOne staff inspected the pipe from the interior to measure the remaining wall thickness (RWT) at very specific locations. Reference the data in Table 6; pipe number 0140 indicated a RWT of 4% at 262 ft. at the 12:30 position and 11% at 260 ft. at the 12:30 position. With this information, WaterOne staff entered the DI pipe and measured the wall thickness utilizing an ultrasonic testing gauge. In order to measure the remaining pipe wall thickness, the cement mortar lining was removed. Figure 8 is a photo of these verification areas in pipe number 0140. The handheld tool in Figure 8 is the ultrasonic testing gauge. Unfortunately, the tool to the right is a pocket knife inserted through the wall of the pipe. The locations of these areas identified by the RFT tool were highly accurate longitudinally and very close to the clock position indicated in the data.

WaterOne staff now felt confident with the data provided and developed a repair schedule. A table was formulated including all corrosion locations of 15% or less RWT. This resulted in the following:

- 0 to 5% RWT → 15 locations
- 6 to 10% RWT → 12 locations
- 11 to 15% RWT → 21 locations

Since there was a short amount of time before this highly critical TM was needed for load season, WaterOne decided to make repairs to areas that had 5% or less RWT. A plan for utilizing clamps in areas of localized corrosion and installing short sections of new pipe for more extensive areas of corrosion was scheduled. In addition, a "test site" was selected to see how the TM performed during load season and try to help determine the rate of corrosion on the remaining pipe. Based on location, low pressure, and low consequence of failure, one 3 ft. area with 2% to 4% RWT was selected as the "test site" and repair work was not performed in this area. The "test site" is electronically monitored for leak detection on a two week cycle.

Figure 8. Verification of Remaining Wall Thickness

REPAIRS AND OBSERVATIONS

As the areas for repairs were excavated, it became apparent that finding the exact location externally was difficult. In many cases a high pressure water jet was used to clean and examine the pipe wall. After extensive examination, the corrosion pits were identified. The pipe had the overall appearance of fair quality, but where the RFT tool found wall thinning there was almost no structural integrity remaining. Figure 9 is a photo of a section of pipe that was removed with multiple corrosion holes. In most locations, pressure from a screw driver or small hammer would reveal quarter to baseball size holes. Repairs were made as follows:

- Clamps (30-inch stainless steel repair bands) = 4
- New Pipe Sections
    - three – 4 ft. sections
    - one – 7 ft. section
    - one – 25 ft. section
    - one – 30 ft. section

- one – 40 ft. section (near shoulder of highway exit ramp)

In addition, the main was retrofitted with cathodic sacrificial anodes in an attempt to add some level of protection to the pipe surface at all repair locations. All blowoff valve assemblies (BOA) were replaced along the pipeline to minimize future problems from corrosion and aide in dewatering the main.

Some observations made during the repairs were: corrosion pits were typically near the 12:00 position on the pipe, approximately one ft. from the end of the pipe segment, and usually under the edge of a road or near the shoulder of pavement. A reason could possibly be the corrosive nature created from the treatment of the roadways during the winter season. This hypothesis seems plausible because in past experiences with other water mains the corrosion locations were usually on the bottom (6:00 position) of the pipe from "hot" soils. Another possible cause of the observed corrosion is a lack of conductivity between joints causing localized electrolysis, but this theory is without a known stray DC current source.

As the final repairs were completed, the pipe sections for the RFT tool insertion/extraction points were put back together. Only one BFV was reinstalled since this pipeline's sole purpose is to feed a reservoir and there are no other connections to it. The TM was disinfected and all excavation sites were restored in preparation for distribution. Lastly, all existing vaults were outfitted with mechanical closures to allow future access if needed.

Figure 9. Pipe Section with Multiple Corrosion Spots

REPLACEMENT SCHEDULE

From the detailed data provided by the CA process utilizing RFT and the known pressures (40 to 110 psi), the probability of failure factor was calculated along the pipeline. Risk values were then calculated as follows:

Risk = Probability of Failure * Consequence of Failure * Reduction Factor

A reduction factor of 75% was used due to redundancy. Based on these risk values and the repairs made to date, WaterOne planned to replace 10% of the 30-inch TM in the winter of 2015-2016, an additional 10% in 2019, and the remaining 80% in 2025. This staggered delay of full replacement led to a \$1.8 million total savings based on the time value of money. The summary of the cost savings is shown in Table 2.

Table 2. Cost Savings for Delaying Replacement of Pipeline

30" DI Transmission Main			
Cash Flow with Delayed Staggered Replacment			
	Discount Rate = Cost of Capital =		4.0%
Year	Full Replacement 100% - 2014	Delayed Staggered Replacement	Delayed Staggered Replacement Present Value
2015	9,700,000	970,000	970,000
2016			
2017			
2018			
2019		970,000	829,160
2020			
2021			
2022			
2023			
2024			
2025		7,760,000	5,242,378
Total Cost	\$ 9,700,000	\$ 9,700,000	\$ 7,041,538
		Savings from Delay	\$ 2,658,462
		Condition Assessment Cost to Contractor	\$ 324,000
		Preparation Costs for Condition Assessment	\$ 408,000
		Repair Costs	\$ 125,000
		Savings from Delay - Costs of CA and Repairs	\$ 1,801,462

CONCLUSION

WaterOne pursued various technologies for the CA of the cement mortar lined metallic pipe. Ultimately, the RFT tool used provided highly accurate, extremely useful information. The CA data was analyzed and aided in assigning risk values along the TM. Based on these risk values along the pipe, a delayed staggered replacement schedule was planned. The successful application of a CA enabled WaterOne to extend the life of the 30-inch TM along with the gained knowledge of the structural integrity of the DI main. WaterOne's hopes of extending the life of the 30-inch water main through strategic delayed replacement based on CA came to fruition resulting in a $1.8 million savings to the utility.

REFERENCES

Pipeline Inspection and Condition Analysis, Corporation (PICA) website and final report to WaterOne. http://www.picacorp.com/services/water-main-inspection.aspx

Developing an Inline Pipe Wall Screening Tool for Assessing and Managing Metallic Pipe

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Abstract

Recent developments in inspection techniques/technologies now make it possible to collect condition data for the entire length of pipeline that can then be evaluated with analytical and engineering techniques to provide a targeted strategy of repair, replacement and management. One specific research and development effort of inline screening technologies began with field trials as part of a 2008 EPA study on innovative condition assessment technologies for water mains. The initial phase of the development of pipe wall assessment (PWA) tools used acoustic pulse technology in qualitative manner to assess the wall strength of a pipeline by determining the change in hoop stiffness over short intervals. On a parallel path, a second PWA technology was developed that measures the change in the self-generated magnetic field produced by ferromagnetic materials in stress. This paper will discuss the development of both technologies.

Condition Assessment of Metallic Pipelines

The management of the buried water infrastructure has seen significant advances over the past decade. Traditionally, pipeline management strategies have focused on establishing risk for a utility's assets based on desktop studies, performing test pitting, collecting limited data on the actual condition, and executing strategic replacement programs. These risk assessments typically use age and the expected life (book value) of an asset to determine the remaining service life of pipelines, which is then a significant factor in the replacement strategy. However, age has been shown to be one of the least reliable predictive factors in pipe failure based on findings by the US EPA, Water Research Foundation, and multiple utilities where data indicates that 70% to 90% of the replaced pipe has remaining life. This realization is causing a shift in industry attitudes away from this traditional approach to pipeline management.

By understanding the risk of the pipelines, the root cause of failures, as well as benefits and limitations of assessment techniques/technologies – a defensible management strategy can be implemented to maintain and extend the life of the assets. Using asset risk to guide the management strategies, owners can ensure they are implementing the right approach, at the right time, with the lowest financial impact.

Traditionally, the assessment of metallic pipelines has been limited to desktop studies evaluating the age and material of the pipeline to determine replacement priorities and when being more proactive, test pits, soil corrosion studies and statistical modeling was used to infer the condition of the entire pipe from a few data points. In the past this was an acceptable and standard method of condition assessment lacking the tools or technologies to inspect the more than a few points along the pipeline. In recent years, indications that these few data points do not provide a suitable replacement strategy as well as advances in technology have made it possible to inspect and collect data over full length of the pipeline allowing owners, operators and engineers data to make more informed and defensible decisions regarding the future management of their assets. Desktop studies and test pits remain an integral part of a condition assessment program of metallic pipe. Using inline inspection technologies can better direct test pits to areas of concern and improve confidence in decisions made. This paper will describe the efforts undertaken to develop the Pipe Wall Assessment (PWA) technology. This technology is an inline screening tool that can be used to identify areas of increased stress on pipe. It can be deployed into a fully operational pipeline on a free-swimming or tethered inspection tool. The results of a PWA inspection identify areas of increased stress on the pipeline which aid in directing the next stage of assessment. In some cases, the use of high resolution tools like MFL or remote field electromagnetics may not be warranted over the entire length of the pipeline. PWA can be used to identify pipe sections or lengths of pipe that have increased stress and aid in directing higher resolution technologies over shorter sections or to direct test pitting. By using PWA to direct test pitting, results of the direct pipe wall measurements collect can be more confidently applied to the overall pipe condition. The likelihood of sampling the pipes in the worst condition on the pipeline is higher giving a higher confidence decisions derived from the inspection results.

ACOUSTIC PWA: THEORY AND BACKGROUND

Acoustic PWA is based on the principle of measuring the velocity of an acoustic wave travelling through the liquid in the pipe. Where the pipe wall is degraded, or less stiff, the wave will travel at a slower rate. This decrease in velocity of the acoustic wave is indicative of reduced hoop stiffness of the pipe. The technology requires an acoustic wave to be induced into the pipe and acoustic sensors to measure the arrival time of that wave over a known distance. The acoustic PWA technologies were based on this theory however, the premise needed further field testing and validation.

SmartBall Acoustic PWA. The first iterations of the technology used the SmartBall® leak and gas pocket detection platform to measure the velocity of acoustic waves in the liquid of the pipe. The tool is free-swimming and negatively buoyant allowing the ball to roll along the bottom of the pipe and collect data relevant to the condition of the pipeline. As such the sensor is always less than one pipe diameter from the pipe wall. The platform has sensors, in its core, recording acoustic data as well as other instrumentation for location and positioning the ball in the pipeline. The device samples data points hundreds of times per second as it rolls through the pipe, gathering data over each pipe section (joint to joint).

To induce acoustic waves into the pipe wall, pulsers were mounted externally at intervals along the pipeline. As the tool travelled through the pipeline, the arrival time of the pulses were recorded on the device's acquisition card. Post processing and analysis of the data used the arrival times and the known distance of the device from the pulsers to calculate the wave velocity over approximately 0.61-meter (2-foot) intervals along the pipeline. Several field trials were run were performed to test the validity and function of the technology. SmartBall PWA was included as part of an EPA Study: *Condition Assessment Technologies for Water Transmission and Distribution System*. The study evaluated different technologies for the assessment of water pipelines and field trials were conducted in July 2009. Five technologies were tested on a 76-year-old, 627-meter (2,057-foot) portion of a 610-mm (24-inch) cast iron pipe in Louisville, Kentucky. After full analysis of the data, shown below in Figure 1, areas of the inspected pipe were identified as anomalous, that is, having reduced wave velocities indicative of degraded pipe wall. Additionally, regular joint signals were visible in the data as well as correlation between known appurtenances (outlet) and increased wave velocity (stiffer section).

As part of the study, 12 pipes were exhumed and pipe wall conditions were fully documented. The Acoustic PWA results roughly correlated and did identifying anomalies on the three most severely damaged exhumed pipes as having reduced stiffness.

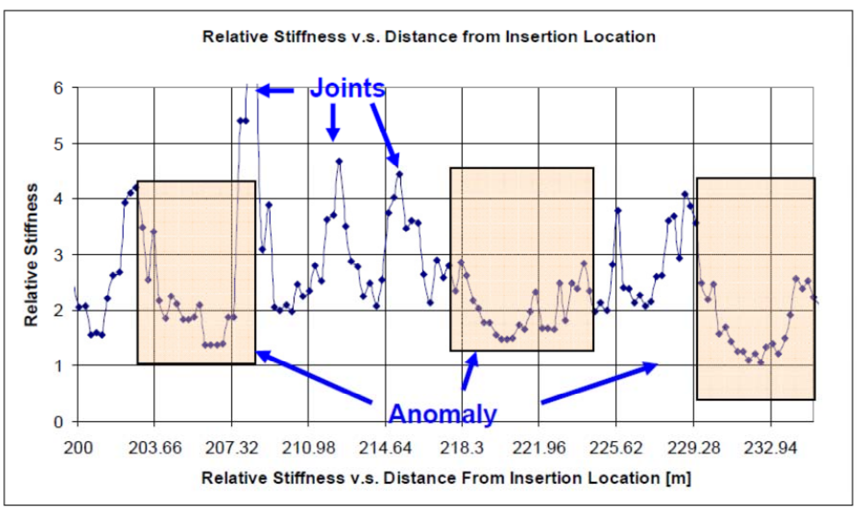

Figure 1: Acoustic PWA Data from SmartBall

Results of the trials in Louisville were promising. Joints were visible, features detectable and areas of decreased wave velocities were evident in the data collected. As more field trials were performed it became evident that results were not always repeatable or reproducible. After considerable effort and investigation it was established that the temperature changes in the water column caused drift in the clock on board the in-line free-swimming tool. Because of the speed at which acoustic wave propagate, the clock accuracy is critical for the measurement of the acoustic wave arrival times recorded. The drift, although on the order of thousandths of a second, introduced enough error into the arrival times of the pulsing making the results unusable. In addition to the clock drift, the position of the ball in the pipeline, although tracked from the surface, is still an estimated distance based on rolling motion and constant flow. These seemingly small inaccuracies in the position of the ball in the pipeline introduced errors into the velocity measurements of the acoustic waves and the results were deemed unacceptable.

An example of the sensitivity of the measurements is shown below. Consider a Class 52 450-mm (18-inch) ductile iron pipe and the simplified velocity equation for an acoustic wave travelling in a medium. The manufacturing tolerance of the ductile iron pipe introduces a variance in the velocity of approximately ±8%. Adding errors from distance and time measurements, over 30 meters (150 feet) with errors of 0.002 seconds and .6 meters (2 feet) yield variances of approximately ±11%.

$$v = \sqrt{\frac{Et}{Dc}}$$

Where:
v = velocity
E = bulk modulus of elasticity
t = wall thickness
D = diameter of the pipeline

Sahara Acoustic PWA. On a parallel and independent path, the Pressure Pipe Inspection Company (PPIC), a separate company at the time, had been testing a similar concept on an acoustic leak detection platform. The tool is a tethered acoustic sensor that can be deployed into a pressurized and flowing pipeline. A parachute pulls the sensor through the pipeline while data is monitored and recorded above ground. For the purposes of pipe wall assessment, an acoustic sensor was mounted externally to the pipeline at a known location. Acoustic waves were introduced into the pipeline by impacting the pipe. The waves travelled through the pipeline and were received by both the reference acoustic sensor mounted externally and by the tethered sensor inside the pipe. Using the arrival times of the acoustic waves and the known position of the sensors, the velocity of the wave was calculated. Data was recorded every 10 meters (30 feet) in the pipe by re-positioning the tethered in-pipe sensor. The average velocity could then be calculated over the 10-meter (30-foot) intervals to screen for sections with lower than nominal velocities that may be degraded.

The tethered in-line tool also participated in the EPA Study field trials in Louisville, Kentucky mentioned earlier. Predicted values for five of the seven validations did not correlate with pipe conditions. It was determined that the variations in the calculated velocities were likely due to inaccuracies in the distances between sensors in combination with measuring velocities over longer distances averaged out any indications of joints and significant pipe wall defects. Additionally any trapped air and gas pockets in the range of the inspection affected the acoustic waves and readings could not be used.

Combining the technologies. In 2010 Pure Technologies acquired PPIC making the combination of the technologies possible. Work with the SmartBall acoustic PWA was halted because of the problems with the clock drift and positional inaccuracies, however, the controlled pulsers were still used to create a controlled acoustic pulse. To deal with the problems that both platforms had with variable distances, two hydrophones were installed on the tethered in-line sensor head at known distance. By fixing the distance between the hydrophones, any error related to distance was removed. Additionally it allowed for readings to be taken over a shorter length, giving the tool a better resolution; on the order of one pipe section.

Field trials were performed using the tethered in-line platform with fixed distance sensors in combination with the external pulsers. The results yielded were very repeatable and indicated the presence of appurtenances on the pipeline, however, because of the averaging of the velocities over the distance between the hydrophone sensors, joints were not visible as they were averaged into the velocity. Detecting the presence of joints, a stiffer part of the pipeline, was a litmus test for the technology. Not detecting the joints sent the team back to the drawing board to further improve the technology.

Sahara II Acoustic PWA. Several physical constraints of the original Sahara system impeded development of the acoustic PWA technology. A new version of the platform was developed that made improvement to the acquisition rates, bandwidth and included multiple hydrophone sensors. Moving from a copper based cable to a fiber optic cable allowed for longer deployments, improved video quality and more data channels to the sensor assembly. Multiple hydrophones allowed for more sophisticated analysis of the arrival times as well as increased ability to distinguish between the different waves that are formed and travel throughout the pipeline and provide better resolution.

The external pulsers were problematic due to access requirements to the pipeline. Additionally, the strength of the pulses received on the varied with the distance from the pulser which created room for further error. For these reasons the pulser was incorporated into the sensor assembly of the new version of the system. This fixed another variable (distance from the pulser source and strength of the pulse received) further removing a possible source of error from the data set collected and made inspection access requirements more management for the pipeline owner. The new pulser was also designed to create the desired wave in the pipeline that would maximize the results found. Different acoustic sources create different waves in the pipeline; some wave forms do not fully develop in the pipe wall and can become a disturbance in the data making the desired wave form harder to identify and analyze.

More field trials were conducted to evaluate the viability of the new tool. The first trials were conducted in Ontario, Canada on a 300-mm (12-inch) ductile iron pipe. The results were the most promising to date. In the data shown below in Figure 3, joints are visible in the section of ductile iron pipe, and the transition to a PVC pipe can be seen as an upward shift in the data, therefore, increased flight times (slower wave velocity) and less stiff material. Additionally, the effect of an air pocket is evident in the data.

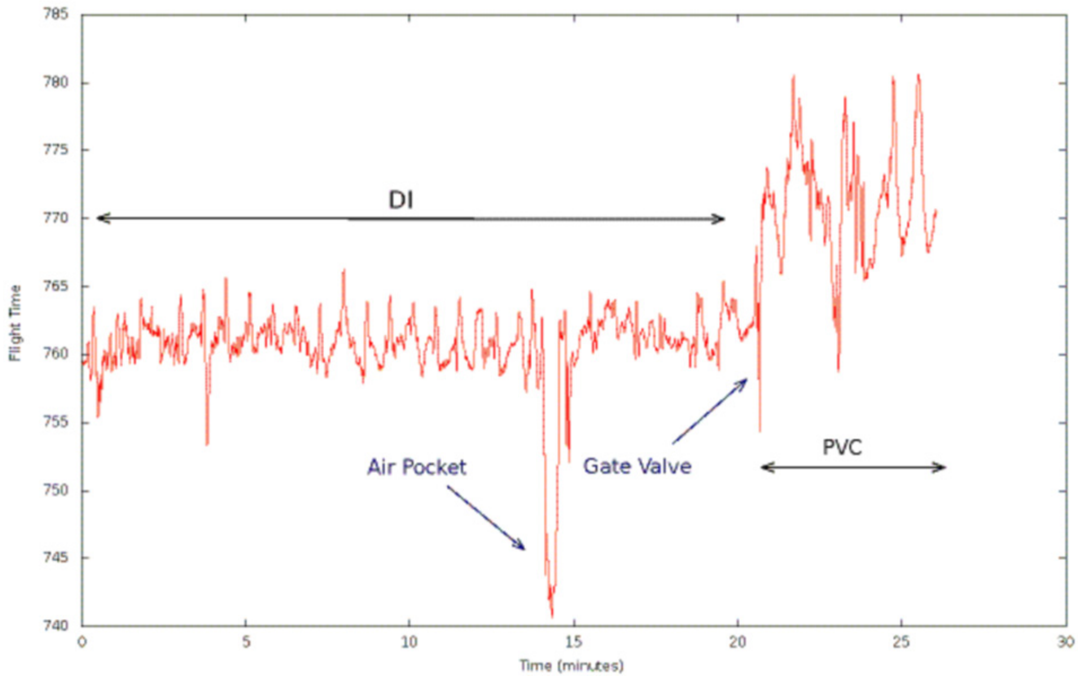

Figure 3: Acoustic PWA from DIP and PVC Pipe

A magnetic flux leakage (MFL) tool was used to inspect the line for wall loss and validate the findings of the PWA technology. Three PWA anomalies were identified on pipe sections where the flight time increased (or the wave velocity decreased) of the acoustic pulses. The MFL tool found two areas of wall loss, which correlated to two of the three locations indicated by the PWA tool. The third PWA anomaly indicated was not investigated further because of access issues. Although no wall loss was found it is conceivable that a manufacturing defect during the casting process could have introduced irregularities in the material properties of the pipe and the tool was detecting a uniformly thinner, but not degraded pipe wall.

A recent field trial was performed on a 450-mm (18-inch) asbestos cement pipe in Australia using the Sahara II PWA technology. Asbestos cement pipe is a good candidate for this method because of its very uniform properties and its mode of failure. A total of 1,325 meters (4,346 feet) of pipeline was inspected in one deployment making it the longest single deployment of the tool to date. The data from 19 pipe sections of the inspection are graphically shown below in Figure 4. Joints are visible in the data at the downward peaks or areas of reduced flight time (increased stiffness). Pipe sections produce relatively repeatable data and obvious outliers are flagged as anomalies. Validations for these results were pending at the time of writing.

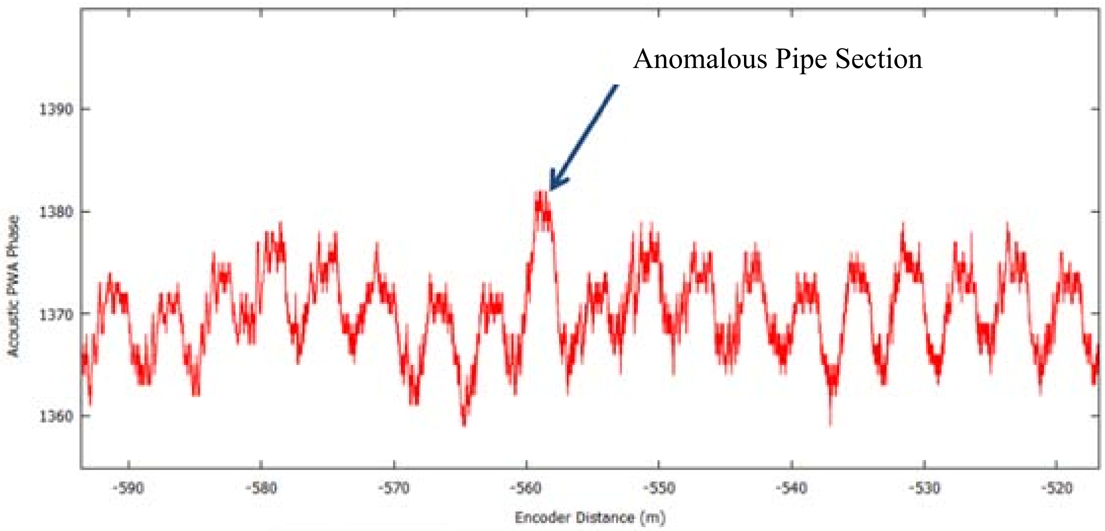

Figure 4: Acoustic Sahara PWA Data from Asbestos Cement Pipe

PWA FOR METALLIC PIPES

In 2013, scientists at Pure Technologies began to investigate a new way to analyze data from the SmartBall that would provide information about the condition of the pipe wall. The SmartBall, since its origins, has had within it several sensors for recording the acoustics within the pipeline, as well as accelerometers, magnetometers and a thermometer. The acoustic sensors were used for identifying leaks and gas pockets in the pipeline while the other sensor were used to position and locate the ball in the pipeline. The techniques for the evaluation is based on the Villari effect; the change in magnetization due to stress in ferromagnetic materials. When ferromagnetic materials are in stress, the magnetic field present is changed. It was believed that the changes in the magnetic field could be measured using the inline condition assessment tools and therefore identify and locate areas in metallic pipes indicative of increased stress.

This method of PWA is particularity interesting in that measuring the stress the pipe wall can be more indicative of condition of the pipeline. Stress in metallic pipe is increased wherever the wall is thinned, where cracks have developed even if they are not through the wall, where the pipe has been damaged or pitted externally or internally, where the pipe is under severe bending, compressive, tensile, or torsional stress, where the original construction of the pipe wall is anomalously thin, or where a pipe is under-designed for its current loading conditions.

SmartBall II PWA. The premise was tested from a large library of data previously collected as the SmartBall platform has always contained magnetometers. Data from previous inspections was reviewed for both repeatability in the data and analyzability. With software improvements to allow for viewing and filtering the data, it became apparent that the data was repeatable from inspection to inspection. Additionally, it was expected that the data produced from similar and undamaged pipe sections would be similar and repeatable. Several sets of data were analyzed as proof of concept of the technology. One data set is shown below in Figure 5 below shows data collected during the EPA Study in Louisville, Kentucky, mentioned earlier. The data response shows repeatability across what was expected to be similar pipe samples; joints are visible showing a large change in the magnetic field. Across the barrel of each pipe there is little change. Anomalies in the data were apparent as seen on Pipe 129 and 130.

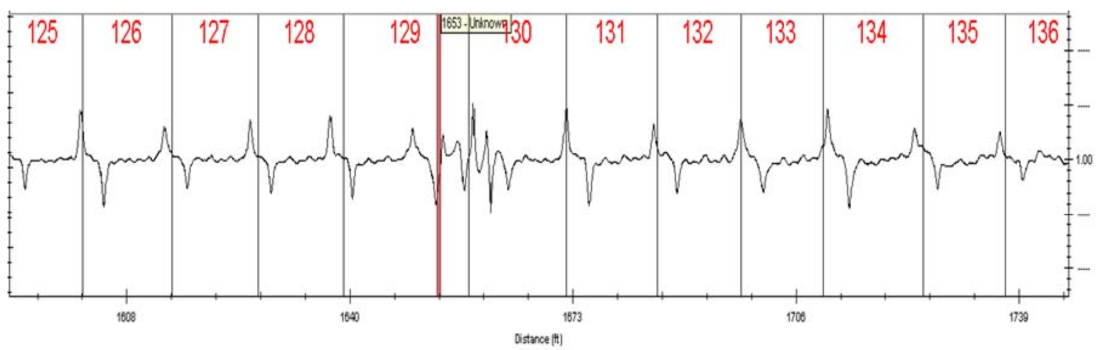

Figure 5: SmartBall PWA Data on Cast Iron Pipe

Initial evaluations of the data were very positive. Correlations were made between pipeline features and anomalies in the PWA data. Material changes in the pipeline, from ductile to cast iron, for example, produced a shift in the baseline of the data. And most promising was the correlation of anomalies to known locations of distressed pipeline. The data from the EPA Study was evaluated and compared with the results of the excavated pipes. As part of the study and validation of the technologies, 12 pipes were excavated and findings were documented. All pipes had some level of pitting and/or corrosion damage. Changes in the magnetic field, or PWA anomalies, were noted on eight of the 12 pipes and more significantly on all three of the pipes noted as having the most severe damage. Figure 6, below, shows the PWA data recorded across Pipe 49, a pipe rated as "most severely" damaged by the study. The photo from EPA study below shows a large grouping of pits with up to 85% wall loss. The PWA data is plotted with a solid red line and is compared to the baseline signal plotted in dashed gray. Changes in the signal (solid red) across this pipe section show a change in the magnetic field, indicative of stress in the pipe wall. In this case, the tool measured the stress created by the pitting, not the actual pits.

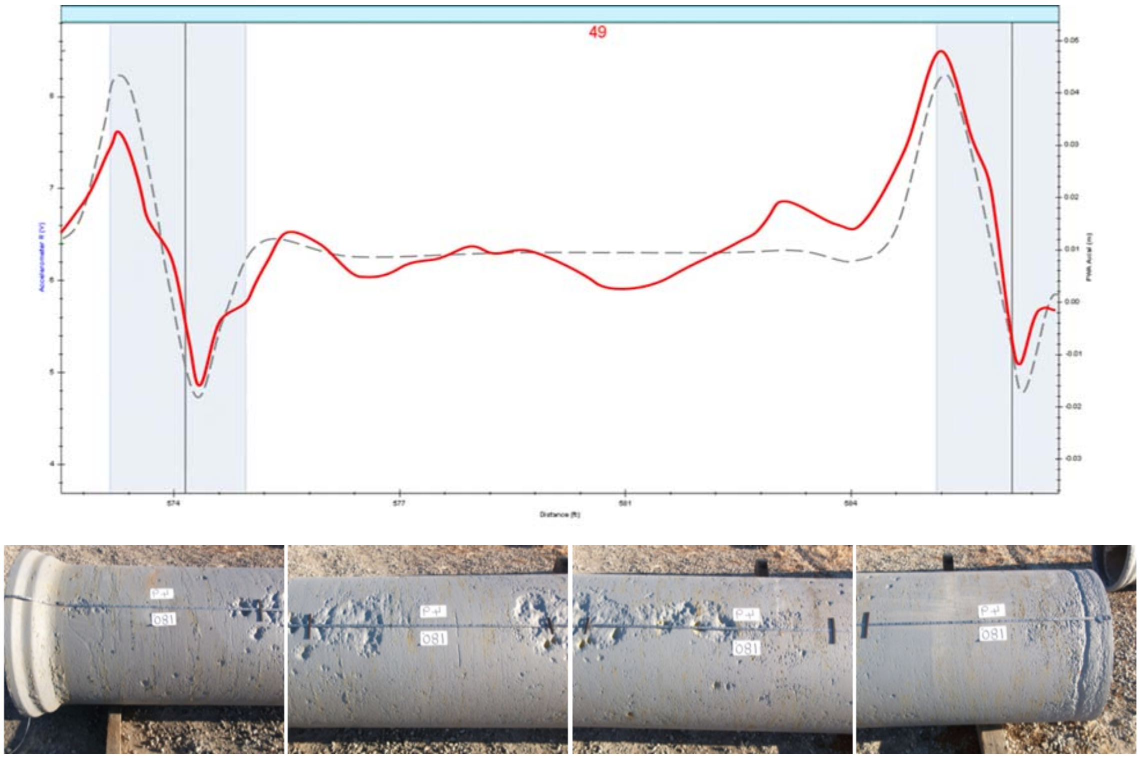

Figure 6: PWA Data and Corresponding Defects on Cast Iron Pipe

The decision was made to proceed with further research and development of the metallic PWA technology after these and other positive results correlating previously collected data to that of know failures, damage and other pipeline features. A second generation mark II SmartBall was developed to enhance PWA capabilities by employing more sensitive instrumentation that detect smaller changes in the magnetic field present in metallic pipes. New sensors allow for algorithms to be applied during analysis process to remove interference caused by irregular rolling of the ball. Additionally, sensors were added to the Sahara II platform to make it capable of collecting PWA information as well.

METALLIC PWA VALIDATIONS

To date over 130 kilometers (80 miles) of pipelines have been inspected using the metallic PWA technology. At the time of writing, validations were in progress on PWA data collected with the Sahara II platform on at 1200-mm (48-inch) cast iron pipe for an owner in the Mid-west. The first of five planned validation sites had been excavated. A section of the original pipe had been removed to provide access for a re-lining project in the 1980's. The pipe was repaired using a 2-meter (6-foot) section of ductile iron, two sleeves and four clamps, additionally bell clamps had been installed at the joints upstream and downstream of the access point as shown below in the photo and diagram in. At the time of the inspections the client was unaware of the exact location of these access points. Although not the intended result of the PWA analysis, the field validation of this point did correlate with the anomaly produced by the short change in material and the presence of extra joints. Further validations are planned with this owner for spring of 2015.

It is expected that several other utilities will be validating PWA results in the first half of 2015 based on the results of PWA inspections. These results will be published and presented as experience allows.

CONCLUSION

The development of new and innovative technologies is a challenging and important undertaking. The PWA technology is an exciting advancement in the field of condition assessment techniques. It will provide owners and operators a screening tool to identify areas of concern along the length of the pipeline by deploying a relatively simple tethered or free-swimming tool without disruption to service. Further testing and direct pipe wall measurements can be focused using the results of the PWA inspection as part of a condition assessment program.

Acoustic PWA has developed extensively over the past six years with many lessons learned along the way. The current version of acoustic PWA has overcome the challenges presented by having sensors distributed over longer distances on the pipeline. Distance errors between sensors ultimately produced unacceptable results from both an accuracy and resolution perspective and drove the design to a fixed distance array of pulsers and sensors. This improved resolution (to pipe joints), sensitivity and additionally, limited the negative effects of gas pockets to only the length of the gas pocket.

In 2013 a new technique was developed to analyze magnetometer data and identify areas of stress on the pipeline. This method of PWA is fundamentally different than acoustic PWA and measures the change in the magnetic field related to changes in the stress in ferromagnetic materials.

Validations of both acoustic PWA and metallic PWA are planned in 2015. Further developments and refinements to the tools, technologies and analysis techniques will inevitably be made in the future.

Pure Technologies would like to thank the Battelle Institute and the EPA for their contributions and continuing efforts to improve and encourage the development of technologies for condition assessment of water and wastewater pipelines through research studies and programs.

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Comprehensive Condition Assessment of Large Diameter Steel Pipe—The Next Chapter in San Diego County Water Authority's Asset Management Program

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Abstract

The San Diego County Water Authority operates and maintains the San Diego region's aqueduct delivery system, which consists of approximately 483 kilometers [300 miles] of large-diameter pipelines, more than 1,400 aqueduct-related structures, and over 100 flow-control facilities. These facilities occupy approximately 567 hectares [1,400 acres] within the Water Authority's right of way, and deliver water for over 3 million residents. This paper will examine some of the steps taken by the Water Authority to continue the development of its asset management program. With many years of focus on managing prestressed concrete cylinder pipe (PCCP), the program is now progressively including other pipe types. Utilizing state-of-the-art technology, the Water Authority is actively including the comprehensive assessment of critical components of its 204 kilometers [127 miles] of welded steel pipe (WSP) and making long-term projections of remaining life based on data gathered. The paper will describe the process of undertaking comprehensive condition assessments using high-resolution magnetic flux leakage technology, lessons learned, and improvements that were made between projects. The paper will also describe how the Water Authority funds such ventures, and how it uses the collected data to determine remaining life in the context of its entire asset management program.

INTRODUCTION

The Water Authority's asset management program was initiated in 2009 to formalize and consolidate a number of asset management efforts being implemented separately (Coghill[1] et al., 2014). This included the Aqueduct Protection Program, which, for 15 years prior, had mainly focused its comprehensive assessment efforts on the 132 kilometers [82 miles] of ageing prestressed concrete cylinder pipe (PCCP) within the Water Authority's entire 483 kilometers [300 miles] of large-diameter pipelines. The program is now progressively including other pipe types. Utilizing state-of-the-art technology, the Water Authority is actively including the comprehensive assessment of critical components of its 204 kilometers [127 miles] of welded steel pipe and making long-term projections of remaining useful life based on data gathered. The decision to undertake a comprehensive condition assessment of welded steel pipe is based on the concept of remaining useful life, which in turn drives a rolling 5-year condition assessment plan.

Useful Life. It can be difficult to accurately predict the lifespan of a pipeline asset. There are numerous factors that can affect a pipeline's lifespan from material composition and construction practices, to environmental conditions and operational influences. Understanding whether a pipeline will attain 90 years or 110 years of life, for example, is nearly impossible to predict with accuracy. The process can be simplified, however, by assuming a specific lifespan and subsequently measuring performance against that initial assumption. Modifications can then be made, based on condition assessments performed as the pipeline ages. It is under this premise that the Water Authority establishes its rolling 5-year condition assessment plan.

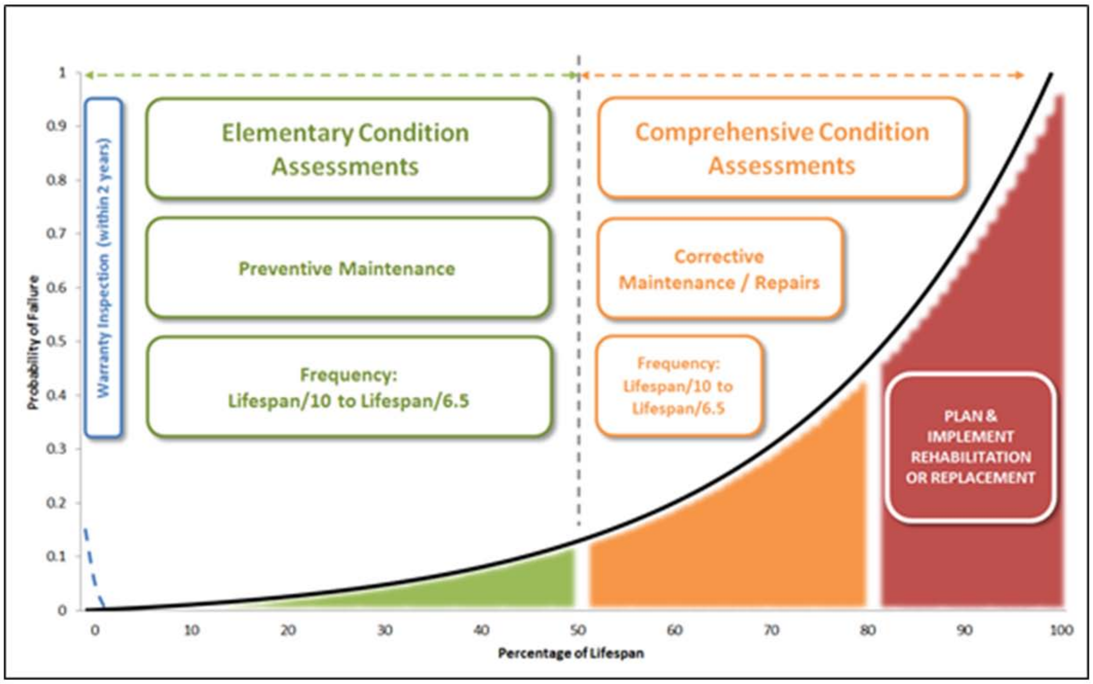

Figure 1. Pipeline Risk Curve with Condition Assessment Milestones

Assuming that a pipeline's probability of failure increases as it ages, as suggested in Figure 1, it seems reasonable that elementary periodic inspections, such as visual inspections, be performed in the early stages of the pipeline's life at, say, 10 to 15 year intervals. This is represented on Figure 1. The Water Authority considers that, based on an assumed full-term useful life (say 100 years), that comprehensive condition assessments should be performed at or near 50 percent of the useful life (i.e. 50 years). Periodic repetition of these condition assessments, on a cycle of 10 to 15 years, will provide a rate-of-change with respect to identifying factors that might be detrimental to the useful life of the pipeline. This is also represented on Figure 1. Ultimately, within the final 20 percent of a pipeline's life, a plan for rehabilitation or replacement needs to be implemented. Throughout the process of evaluating and re-evaluating the rate-of-change of a pipeline asset, it is entirely possible that the initial assumption of an asset's useful life might be extended. Perhaps the rate-of-change is low, or non-existent, or perhaps local repair efforts might maintain or extend the assumed useful life with minimal effort. Alternatively, implementing comprehensive condition assessments at the 50 percent stage might identify significant rate-of-change, resulting in a reduced useful life that can be managed at the right time.

Other factors, beyond anticipated useful life alone, may also play a part in the planning of a comprehensive condition assessment. The Water Authority is in the final stages of implementing a regional desalination plant. The take-or-pay agreement with the plant's owners and operators means that a portion of the existing pipelines needs to perform adequately for the term of the agreement and beyond, without unscheduled interruption. This played a part in scheduling a comprehensive condition assessment for the portion of pipeline dedicated to the conveyance of desalinated water ahead of its 50 percent useful life milestone.

Funding. It is worth noting that the Water Authority considers the data obtained through comprehensive condition assessments, such as electromagnetic or ultrasonic scanning, to be actionable data. That is, the data allows the Water Authority to make informed decisions with respect to localized repair needs and to global useful life predictions and amendments. As this is directly connected to the asset lifespan, the costs of performing comprehensive condition assessments can be capitalized against the asset. The funding mechanism is through the capital funding process where the funds are borrowed through long-term (typically 30-year) municipal bonds. This supports the Water Authority's desire for long-range financial planning and rate stabilization.

COMPREHENSIVE CONDITION ASSESSMENT

Based on estimated remaining useful life, the Water Authority established a need to comprehensively assess the condition of approximately 34 kilometers [21 miles] of welded steel pipe within a 2-year period. This was split between two pipelines, Pipeline 4 (18 kilometers [11 miles]) and Pipeline 3 (16 kilometers [10 miles]).

Contractor Selection. In order to evaluate alternative technologies in a competitive environment, a Request for Proposals was published in March 2013. An outreach effort was conducted to gather interest, which resulted in the pre-proposal meeting being attended by 16 different firms, with a total of 96 firms downloading the Request for Proposals and associated documents. In May of 2013, just one proposal was received, proposing the use of high resolution magnetic flux leakage (MFL) technology. It was established, from follow-up research, that firms representing other technologies did not propose as they could either not compete with the high-resolution output of MFL, its accuracy, or complete the assessments within the timeframes required.

The Water Authority at that point had previous experience with MFL technology, being implemented on an 8 kilometer [5-mile] stretch of 1.8 meter [72-inch] diameter pipeline. However, that work was completed during a pipeline outage, which did not have significant time restraints so an element of trial-and-error was permissible. The scope of work being proposed for each of the two new phases of work, limited the timeframe to 21 days each. This was identified as a significant risk by the Water Authority, so only the first phase of work was authorized (Pipeline 4) following a short period of negotiation and planning. Also, the Contractor elected to utilize some spare pipe stock in order to conduct a trial setup and test runs of data gathering prior to the pipeline shutdown, see Figure 2.

Figure 2. Trial Setup of MFL Tool

The Water Authority decided that Pipeline 3 would only be awarded following successful completion of Pipeline 4. This, as it turned out, also offered the opportunity for lessons learned during the Pipeline 4 operation to be implemented and certain other conditions to be met ahead of the Pipeline 3 operation. This is further explained later in this paper.

PIPELINE 4 PROJECT (October 2013)

The Pipeline 4 project consists of approximately 18 kilometers [11 miles] of 2.4 meter [96-inch] diameter welded steel pipe with a cement mortar lining. The MFL scanning effort was to take place within a 21-day shutdown of the pipeline. It is worth noting that this portion of Pipeline 4 is a treated water line, so every component, and every item of personnel footwear, had to be disinfected prior to entering the pipeline. This was achieved using a bleach solution and hand-held sprayers and/or foot troughs.

Challenges. Aside from the relatively tight schedule, a number of challenges lay ahead for the project team to plan around. For example, a 365-meter [1,200 foot] section of the pipeline which lay beneath a local amenity lake could not be fully drained over fear of flotation forces acting on the pipe. The tool took 3 days to assemble within the pipeline as each component was required to fit through a 0.5 meter [20-inch] manway. Given this, the contractor was required to strategize a means of ensuring the MFL tool could pass through the undrained section of pipeline. This was achieved by utilizing a remotely operated vehicle to pull a winch rope through the pipeline underwater. The contractor utilized the vehicle to visually look for any debris build up, and SONAR was used to detect any pipe ovality issues.

The steep terrain along the alignment of the pipeline was another challenge, overcome by adopting an external winch crew to pull the MFL tool where supporting equipment was unable to be self-propelled. Grades of up to 31 degrees were negotiated.

Figure 3. Winching of MFL tool

One further challenge that was planned for was the verification of data that would be received during the inspection. It was the Water Authority's intention to verify sample data gathered using MFL by utilizing a different technology. This came in the form of ultrasonic testing (UT) by removing the cement mortar lining in localized areas and scanning to verify the anomalies detected by MFL. This proved to be a valuable exercise in supporting the high level of confidence that was attained in the MFL-presented data.

Limitations. In addition to the challenges of the project, the MFL tool had inherent limitations due to the size and complexity of the tool. These limitations were well known prior to the inspection and contingencies were put in place to address them. However, as with any complex project, other limitations made themselves apparent as the project progressed. A brief summary of the limitations are explained below:

- Tool Speed: One of the known limitations was the maximum speed at which the MFL tool can traverse the steep areas along the pipeline. For the majority of the inspection, the tool relies on two electric ATVs inside the pipeline attached to the tool through a hitch system which produces a speed of approximately 0.4 meters per second [1.5 feet per second]. However, on steep slopes, the tool needs to be assisted with external winching crews. This produces a speed of approximately 0.1 meters per second [0.4 feet per second] and dramatically slows the tool productivity. In addition to the slower winch speed on the steeper slopes, the team also had to take into account the extra time to set up the winching crews into the overall schedule.

- Data Download Times: Data was retrieved from the MFL tool at a 1:1 ratio of download time to inspection time. That is, an 8 hour day of inspection required 8 hours to download data. In addition to a lengthy download time, the integrity of the data could not be confirmed until it was downloaded and examined by the data analyst. This long download time did impact the schedule.

- Hitch Connection: The hitch system connecting the MFL tool to the two electric ATVs was a rigid connection (similar to a ball trailer hitch) designed to center the MFL tool in the pipeline. It was important to keep the tool centered to ensure that the magnets were the proper distance away from the steel plate. This design had limitations, discussed further below.

Actuals. The MFL inspection of Pipeline 4 covered 18 kilometers [11 miles] and occurred over an 18 day period from October 8 to 25, 2013. This time period included 3 days to build the MFL tool in the pipeline, 9 days of MFL inspection, 5 days where no data was collected, and 1 day to breakdown the MFL tool and remove it from the pipeline.

- <u>Tool Speed:</u> The overall speed of the MFL tool was 0.09 meters per second [0.3 feet per second]. This slower speed than anticipated was due to the ATVs overheating and the steep slopes.

- <u>Data Download Times:</u> In order to retrieve data from the MFL tool each day, the tool needed to be shut down before the data canister could be removed from inside the pipe. Following the lengthy download process described in the Limitations section above, the analyst could then begin the preliminary analysis looking for anomalies greater than 50% wall loss so that they could be reported to the Water Authority. This method of data retrieval created a large time gap between the inspection of the pipeline and notification of anomalies (greater than 50%) present. The preliminary analysis became a week behind schedule and daily updates of the preliminary analysis were often not completed.

- <u>Hitch Connection:</u> The hitch system as designed created difficulties while navigating compound bends and the hitch repeatedly disconnected from the ATVs. This rigid hitch connection also required the front and rear ATVs to match their speed in order to distribute the load of the MFL tool equally. The ATVs did not have a speedometer or load indicator, which made matching speed difficult. The extra load placed on the faster ATV would cause it to overheat and shutdown. Inspection could not resume until the ATV cooled down. Field crews modified and mounted electric radiator fans to help cool the electric motors of the ATVs. While this did help, on long runs the ATVs would still overheat and thus impact the schedule.

Following the completion of the MFL data analysis, 1,107 locations of wall loss were identified. Of the 1,674 pipe sections of Pipeline 4 inspected, 1,078 pipe sections (64%) had no detectable damage. Within the 596 pipe sections that did have damage, 7 areas of wall loss greater than 60% wall loss were detected (see Figure 4).

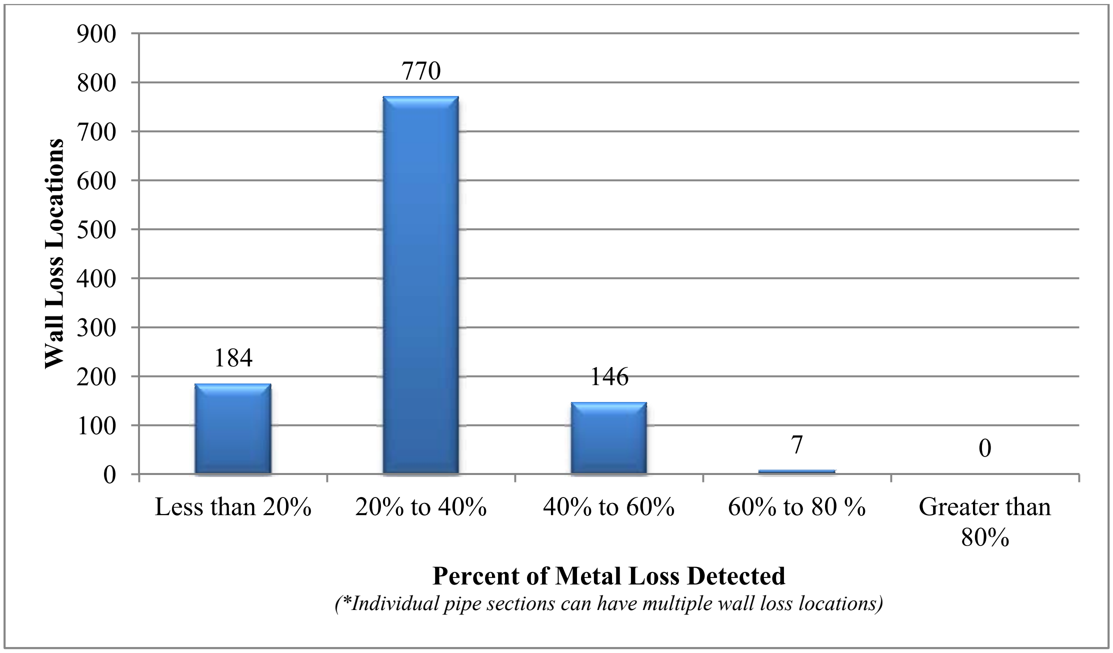

Figure 4. Pipeline 4 Percent Wall Loss Per Anomaly

Lessons Learned. After the Pipeline 4 project was completed, the Water Authority and contractor Pure Technologies met to discuss lessons learned on the project and the corrective measures that needed to take place before the Pipeline 3 project could move forward. The team decided that there were 4 major items that needed to be corrected. These are listed below with the corrective action taken.

Table 1. Improvements from Lessons Learned Meeting

Issue	Corrective Action Taken
Time taken to download data from the tool exceeded the MFL scan time	Data is now written directly to a removable hard drive that can be removed from inside the pipe.
The time taken to analyze data to obtain preliminary condition assessment results was excessive	With the download time now completely removed from the equation, and only looking at wall loss areas greater than 50%, this issue was corrected.
Failures of the power module/battery pack (for data canister)	Holes in the battery canisters were machined to correct the shorting of the batteries to the canister.
Resource Utilization, long shifts over multiple days decreased productivity	Staffing plan was developed to address resource utilization with a mandatory day off for all staff in the middle of the inspection.

PIPELINE 3 PROJECT (October – November 2014)

The Pipeline 3 project consisted of approximately 16 kilometers [10 miles] of predominantly 1.9 meter [75-inch] diameter welded steel pipe with a cement mortar lining. The MFL scanning effort was to take place within a 21-day shutdown of the pipeline.

Challenges. Similar to the Pipeline 4 project, the Pipeline 3 project included significantly steep terrain. What compounded this was the fact that the pipeline is untreated, and so is lined with a slippery bio-film that requires drying out to gain traction. This was achieved using forced-air blowers installed as soon as the pipeline was drained at multiple locations. The drying process was successfully achieved within the 3-day timeframe it took to assemble the MFL tool, so it was not an impact to the overall schedule.

A potential threat to schedule, however, was the need to adjust the MFL tool size at several locations to account for pipeline diameter changes. These existed at high-pressure points (around 28 bar [400 psi]). The pipeline reduced in diameter at these locations to 1.8 meters [72-inch].

Another known challenge was the existence of three full-size wye connections. The MFL tool had never previously passed a full-size wye connection. These connections are an obstacle because the MFL tool is designed to center itself in the pipe. With no pipe wall on one side of the tool, there exists a possibility for the MFL tool to push itself into the open space of the full size wye.

Preliminary Data Analysis. The preliminary data analysis was a major focus during the MFL inspection of Pipeline 3. Improvements to the MFL tool, as discussed above, greatly increased the ability to deliver this preliminary analysis on time. A real-time monitor on the MFL tool alerted crew members as to the quality of data as it was recorded throughout the day. Additionally, data was recorded to a removable hard drive which could be removed from the data canister and viewed on a laptop onsite for data quality. This instant retrieval of data allowed for data to be retrieved twice per day.

Following the third day of inspection, the Water Authority was provided with actionable data in the form of a table of pipes identifying dimensions and locations (longitudinal and radial) of areas with metal loss greater than 50%. This preliminary analysis was provided at a rate of 1.6 km (1 mile) per 24 hour period and included in a daily update email at the end of each inspection day. The preliminary analysis was completed five days after the completion of the MFL inspection.

It should be noted that the preliminary data analysis has its own limitations due to the amount of time allowed for analysis. An additional 34 anomalies with metal loss greater than 50% wall thickness were identified in the final analysis. These anomalies may have been missed during the preliminary analysis due to their length, proximity to a joint, or lift-off of the hall-effect sensor from the pipe wall.

Verification. A significant component of the work for the Water Authority was the effort to verify the MFL data. It is the Water Authority's intention to verify the technology used on every project involving comprehensive condition assessment, where feasible. As the procedure for preliminary analysis had improved, data was presented early in the pipeline shutdown which enabled the verification of MFL data. This was achieved by chipping away patches of cement mortar lining at select locations, and scanning the steel wall using an ultrasonic flaw detector. This operation was conducted by Water Authority staff. To back up this effort, the Water Authority employed the services of an independent Level III UT specialist for one day to confirm the Water Authority's findings at several locations.

Improvements. As mentioned previously, the award of the Pipeline 3 work was dependent upon improvements that were identified during the Pipeline 4 project. These improvements were to be made by the contractor, and successfully demonstrated, prior to contract award and are briefly discussed in Table 1 above. The most impactful improvement to the tool was the ability to extract the data immediately which saved the team valuable time.

Another impactful improvement that was added was a set of wheelie bars to each ATV as a mechanical means of keeping the wheels in contact with the pipe wall. The previous method had relied on adding weight to the ATVs which caused some damage and cracking to cement mortar lining on previous inspections. The front ATV had a bar on the front with a wheel at the 12 o'clock position in order to keep the front wheels down. The rear ATV had a bar coming off the rear with a wheel at the 6 o'clock position. Only a small amount of weight had to be added to help with traction in slippery areas.

Communication was improved using daily updates to the Water Authority each evening. This communication included the progress of the tool, comments on the day's activities, and plans for the following day. The Water Authority would discuss the updates the following morning before the next day of work and could better utilize their resources to put them where they are needed. This improved communication combined with the addition of strict time schedules, working normal hours and the mandatory day off resulted in a safer project for all.

Actuals. The MFL inspection of Pipeline 3 covered 16 kilometers meters [10 miles] and occurred over a 16 day period from October 22 to November 6, 2014. This time period includes 3 days to build the MFL tool in the pipeline, 11 days of MFL inspection, 1 day no data was collected due to a planned safety stand-down, and 1 day to breakdown the MFL tool and remove it from the pipeline. Some notable improvements, milestones and a few challenges were:

- No Down Days: Crews were working by 7:30 AM each day and out of the pipeline at 5:30 PM each evening. The crews completed the longest single day of inspection out of any MFL inspection while staying within this schedule.

- Better Planning: A large amount of time was spent prior to the project creating an inspection schedule and knowing where certain objectives would happen. Everyone knew where the MFL tool should be and how far it should get by the end of the day. This allowed the team to stay ahead of schedule the entire inspection using 55 open access points and 24 planned winches.

- Final Data Analysis: The MFL tool produces an enormous amount of data that requires a very in-depth and time consuming analysis. The contractor is continuing to improve the data analysis process to cut down the amount of time required.

Following the completion of the MFL data analysis, 859 locations of wall loss were identified. Of the 1,837 pipe sections of Pipeline 3 inspected, 1,234 pipe sections (67%) had no detectable damage. Within the 603 pipe sections that did have damage, 27 areas of wall loss greater than 60% wall loss were detected (see Figure 5).

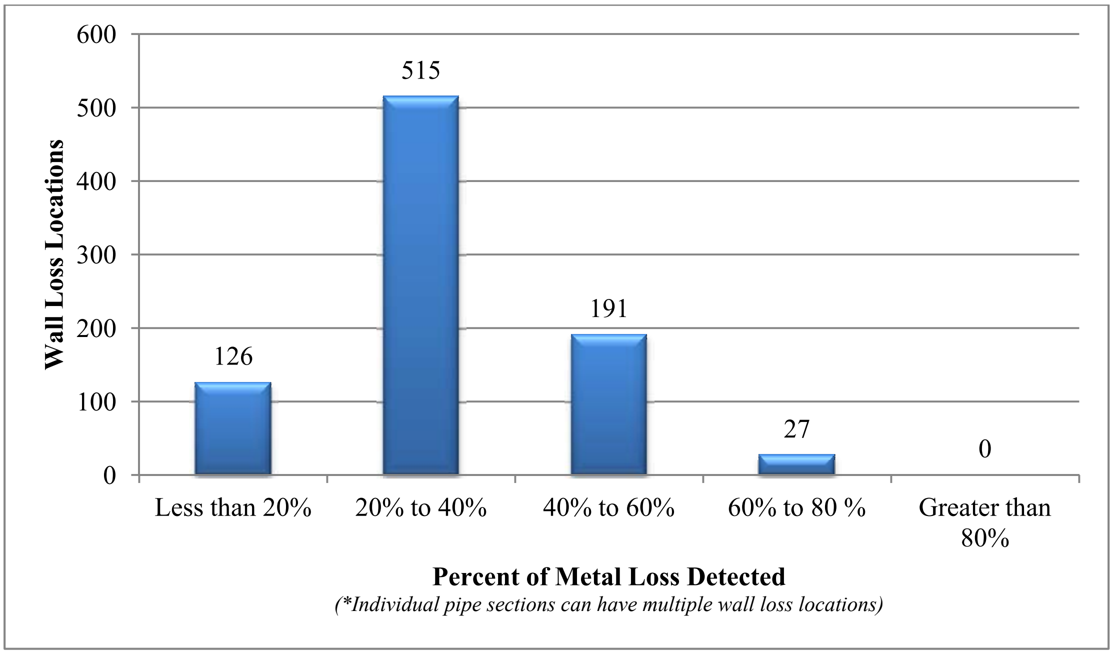

Figure 5. Pipeline 3 Percent Wall Loss Per Anomaly

Pipeline Repairs. With the MFL scanning successfully completed ahead of schedule, and preliminary data submitted on schedule, the Water Authority was able to identify four locations for steel wall repair within the pipeline shutdown window. These repairs were based on verification of the existence of corrosion, and the percentage of wall loss measured by UT scanning being greater than 70%. Repairs were performed by the addition of steel plate patches welded to the inner surface of the pipe to provide additional structural strength and pipeline longevity. An example is shown in Figure 6.

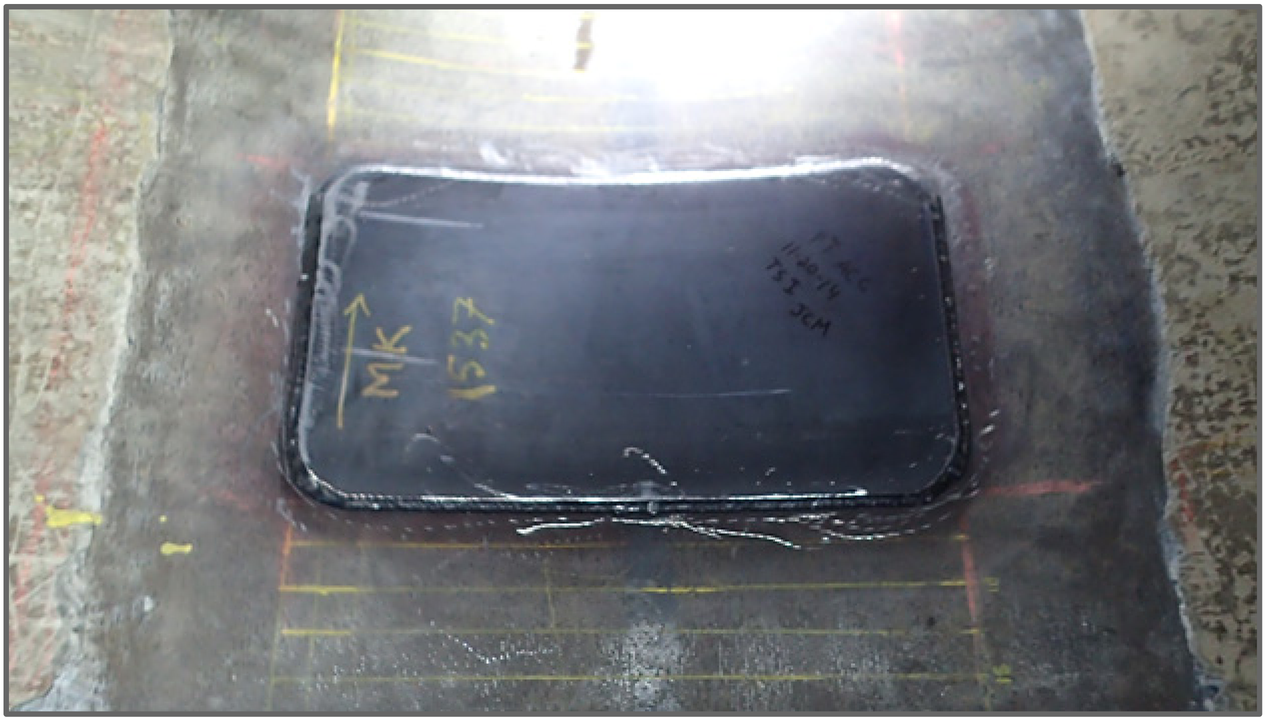

Figure 6. Steel Patch Repair

CONCLUSION

As discussed earlier, the ability to enforce improvements of the MFL tool between projects permitted the reduction in a number of risks associated with conducting an MFL assessment of a large diameter welded steel pipe. This included a significant reduction in work days required to complete the effort, less impact to the personnel performing the work, a quicker turnaround of preliminary analysis, and the ability to perform steel patch repairs during the same pipeline shutdown. These improvements benefited the contractor and the Water Authority and are a testament to the willingness to continually improve, maintain open and transparent communications, and the understanding of each other's ultimate goals. Currently, the Water Authority has successfully conducted the comprehensive condition assessment of 42 kilometers [26 miles] of large diameter welded steel pipe, which is 60 percent of the total distance planned within the current 5-year outlook.

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The Case for Large Diameter Pipeline Condition Assessment

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Abstract

Condition assessment projects for large diameter pipelines can be very expensive andoperationally complicated. Costs not only include the condition assessment effort, but additional costs for preparation of the project, pipeline shutdown/isolation, engineering, coordination of the project with the public, and non-revenue discharged water. Owners are finding condition assessment projects expensive and difficult to justify to their board of directors and governing bodies. This paper will cover the San Diego County Water Authority's typical large diameter pipeline condition assessment costs inclusive of all aspects of the project. In addition, the paper will describe the Water Authority's justification for condition assessment projects with four main cost benefits: preventing failures with a proactiverepair approach, maintaining a reliable water supply, extending the life of pipelines, and efficient useof replacement and repair funds.

BACKGROUND

The San Diego County Water Authority (Water Authority) is a public agency serving the San Diego region as a wholesale supplier of water from the Colorado River and Northern California. The Water Authority's mission is to provide a safe and reliable supply of water to its 24 member agencies serving the San Diego region. The Water Authority operates and maintains 300-miles [480 km] of large diameter (48-inch [1220-mm] to 108-inch [2740-mm]) welded steel pipe, prestressed concrete cylinder pipe (PCCP), bar-wrapped concrete cylinder pipe, and reinforced concrete (cylinder and non-cylinder) pipe.

The Water Authority has been assessing the condition of its large diameter pipelines since the early 1980s. See Figure 1 for a timeline of the Water Authority's condition assessment history. Prompted by failures, the Water Authority began focusing on problematic PCCP areas with targeted soil potential surveys, pipe excavations, and destructive testing in 1981. The investigations lead to a targeted rehabilitation effort for 5-miles [8-km] of pipeline. In 1992, the Water Authority's Board established the Aqueduct Protection Program (Galleher 2007) and developed a comprehensive plan for condition assessment and estimation of service lives for all pipelines. The estimation of service lives was based on several factors including data from internal pipeline inspections and PCCP sounding.

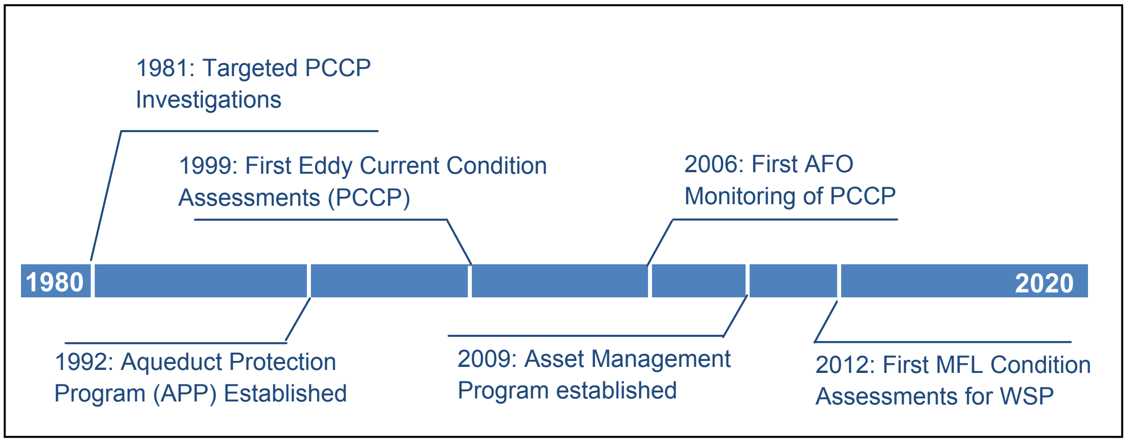

Figure 1. Water Authority Condition Assessment Timeline

In less than eight years, the Aqueduct Protection Program had completed internal inspections and sounding all 82-miles [132 km] of PCCP in the Water Authority's system. In 1999, the Water Authority supplemented visual and sounding condition assessment efforts with eddy current electromagnetic inspection. The Water Authority was able to use this data to better understand pipe conditions and make better decisions on repair and replacement of PCCP. To understand the real-time condition of PCCP, the Water Authority began the first installation of several Acoustic Fiber Optic (AFO) monitoring systems in 2006. These systems generate condition data that helps better understand the changing condition of PCCP with time.

In order to better formalize the program and evaluate all assets, the Water Authority established a formal Asset Management Program in 2009. The Program implements several industry best practices including a comprehensive plan for on-going condition assessment. Data generated from condition assessment is evaluated and used to prioritize and schedule repairs/replacements and adjust the expected service life of the assets. The team uses condition assessment data to estimate remaining life and determine the probability of failure which drives the repair and replacement schedule for each asset or group of assets.

A majority of the effort was focused on PCCP, but in 2012 the Water Authority expanded detailed condition assessments to Welded Steel Pipelines (WSP). At that time, the Water Authority used Magnetic Flux Leakage (MFL) on a critical stretch of pipelines (5-miles [8 km]). Since then, over 26-miles [42 km] or 22-percent of WSP has been assessed using MFL.

CONDITION ASSESSMENT

The Water Authority currently has experience with various types of condition assessments shown in Table 1. Table 1 represents currently accepted methods by the Water Authority to assess large diameter pipeline condition. The data confidence represents the level of confidence the Water Authority has that the condition assessment effort is providing comprehensive data on the condition of the asset. These confidence ratings are based on experience with the technologies and represent the ability to find true indicators of the condition of the pipe. For example, during an internal visual inspection there could be problems or indicators that are not seen, such as external corrosion or wire breaks. Although a visual inspection may reveal some information on the condition of the pipe, there may be more indicators that cannot be seen, therefore the data confidence is lower.

Table 1. Water Authority Condition Assessment Types

Type	Pipe Type	Data Confidence
Visual – Internal	All	Low
Sounding – Internal*	PCCP	Low
RFEC	PCCP	Medium/High
AFO	PCCP	High
MFL	WSP	High
Leak Detection	Bar Wrapped Concrete Cylinder Pipe	Low

* Only used for targeted areas of concern

For the direct costs of the condition assessment effort (not the planning, shutdown, preparation or additional costs), Table 2 shows typical costs incurred by the Water Authority.

Table 2. Typical Water Authority Direct Condition Assessment Costs

Type	Cost per mile	Mobilization Costs	Monitoring Costs (/mile/year)	Installation Costs per mile
Visual – Internal*	$5,000			
Sounding – Internal*	$10,000			
RFEC	$20,000	$35,000		
AFO**		$50,000	$35,000	$140,000
MFL	$120,000	$20,000		
Leak Detection	$15,000	$25,000		

* Water Authority Labor Only
** Monitoring costs include leased data acquisition system

These are typical costs and can vary based on the actual conditions and special considerations of each project.

Operational Complications. All of the condition assessment methods currently used by the Water Authority, except for leak detection, require the pipeline to be out of operation, or shut down, and drained. The planning/scheduling and implementation of operational shutdowns can be complicated. The Water Authority is not unique, for example other agencies including the City of Phoenix (French 2014), the City of Houston (Henderson et al. 2010), and the Metropolitan Water District of Southern California (McReynolds et al. 2014) plan well in advance for pipeline shutdowns and have complicated operational constraints. Factors that are considered in the planning and cost estimating for each shutdown include:

1. Disruption of service to customers
2. Reconfiguration of water system and new operating conditions
3. Isolation of pipelines (valves, blind flanges, air gaps, removal of valves) and safety lockout/tagout requirements for entry into pipeline
4. Coordination with member agencies (customers), contractors, and regulatory agencies
5. Repairs, maintenance, and inspections required for connected facilities to take advantage of the out of service condition
6. Agreements and contracts for service and water supply
7. Storage of water and banking to sustain service during service disruptions
8. Environmental and water quality such as water discharge requirements
9. Pipeline disinfection

Additional Condition Assessment Costs. In addition to the direct costs for condition assessment activities, the additional operational and planning costs are significant. The Water Authority typically budgets and tracks the following costs for each condition assessment effort:

<u>Preparation</u>

1. Planning labor for coordination with others and safety lockout/tagout requirements
2. Public and right-of-way coordination labor
3. Grading access roads and staging areas for vehicle access(when applicable)
4. Checking and preparing pipeline access structures and exercising flange connection bolts

<u>Execution</u>

5. Shutdown Labor and benefits for draining, opening, closing, and refilling pipeline including overtime
6. Traffic Control (when applicable)
7. Materials such as replacement flange bolts, nuts, and gaskets
8. Equipment rentals such as cranes, air blowers, generators, portable toilets, and water discharge treatment
9. Non-Revenue discharged water

The additional costs for condition assessment are significant. The Water Authority has found costs can range as shown on Table 3.

Table 3. Typical Water Authority Additional Condition Assessment Costs

Base Cost (1 mile)	Additional per Mile Cost (Low Range)	Additional per Mile Cost (High Range)
$100,000	$55,000	$75,000

Generally there is a base cost for the initial shutdown preparation and work for the first mile. Then for any additional miles, the cost ranged as shown on Table 3. The base costs are based on past shutdowns and the per mile costs are based on two recent Water Authority shutdowns for a 72-inch [1830-mm] and 96-inch [2440-mm] diameter pipeline, both 10-miles [16 km] to 11-miles [18 km] in length.

JUSTIFICATION

The Water Authority justifies the significant cost of condition assessment activities in four main ways. Condition Assessment efforts help the Water Authority:

1. Prevent failures and public harm, environmental impacts, and repair costs
2. Prevent failures and unplanned outages to maintain a reliable water supply to customers
3. Extend the life of pipelines and realize savings by deferring repair/replacement
4. Spending repair/replacement costs efficiently on pipelines that truly need repair/replacement at the right time

The Water Authority focused on protecting public safety, preventing environmental impacts and avoiding unnecessary repair costs by preventing pipeline failures. In the past 50 years the Water Authority has had nine major large diameter pipeline failures (Faber 2014). Fortunately there have been no public safety impacts due to these failures. However, the consequences of these failures have been high and include repair costs, mitigation costs, lost water, property damage, customer service interruptions, negative public perception of the agency, and litigation. In the last 10 years, failures were in rural areas and impacted the environment. The costs ranged from $1.2 to $2.9 million dollars per failure. The Water Authority estimates a failure in an urban area could impact public safety and exceed $10 million dollars in repair costs. These costs and risks are substantial and the operation of the wholesale system for San Diego County represents a critical responsibility to maintain reliability and prevent failures. Comprehensive condition assessments have helped the Water Authority mitigate the risk of failures and helped maintain a reliable water supply to our Member Agencies. Although the cost of condition assessment is significant the avoided cost of a single failure and the negative perception can be sizable compared to the cost of a condition assessment effort.

Another savings for the Water Authority is the ability to extend the planned life of a pipeline asset and repair/replacement of the pipeline at the right time. In general the Water Authority considers the design life of WSP to be 100 years and a PCCP to be 60 years. With no condition data to justify a longer life and not enough information on the actual condition of the asset, past thinking was to plan to replace or repair these assets at the end of their design life. However, using data from a comprehensive condition assessment combined with real-time AFO monitoring, the Water Authority can extend the expected life of the pipeline, deferring repair and replacement costs and getting the maximum life out of existing assets. The Water Authority successfully uses condition assessment data to be informed about the condition of pipelines and determine a revised service life. In addition, the data from a comprehensive condition assessment and active monitoring can also identify pipelines that will not be able to meet their expected life. Then a sustainable repair or replacement plan can be implemented in advance of significant deterioration. This proactive approach is better than a reactionary or mandated plan because it is less costly and future costs are more predictable over time.

Finally, the Water Authority can justify efficient spending on proactive repair and replacement costs for pipelines based on condition assessment data that drives a risk based approach (Coghill et al 2014). This approach provides confidence for the Board and rate payers that the Water Authority is spending repair and replacement funds on the right assets at the right time. For example, the original repair plan for PCCP was to rehabilitate all the PCCP in the system by 2020. However based on condition data, specifically from the real-time AFO monitoring system, there were several pipelines that did not need immediate or short-term repairs. This led to the deferral of over $200 million of capital spending and more targeted repairs rather than a comprehensive rehabilitation for some pipelines. This also extended the life cycle and helped realize the full life of the asset.

To illustrate the justification for condition assessment, the Water Authority conducted a simple cost-benefit comparison for a future MFL condition assessment project. The comparison is for a 10-mile steel pipeline [16 km] over a 10 year term. Project costs were calculated based on past projects and include the agency costs for preparation, draining, access and filling the pipelines. See Table 2 and Table 3 for more information. Based on past experience, the Water Authority assumed that as a result of this condition assessment, four locations of significant corrosion (greater than 70-percent steel wall loss) would be identified and repaired by welding internal steel patches. For prevention of a pipeline leak, the low (rural) repair costs were based on fixing four leaks over 10 years, each at a cost of $300k which includes a pipeline shutdown, draining, access, repair, filling and property damage/claims due to leaks. The high estimate of $6 million is due to an estimated $1.2M for each leak in additional property damage and claims impacting several urban infrastructure including structures, businesses, and other utilities. Proactive repairs based on an MFL condition assessment would have a benefit of avoided repair and damage costs ranging from $1.2M to $6M over a 10 year period.

The life extension benefits are based on the estimated cost of constructing a replacement pipeline, which would last 100 years, with new construction costs of $88M in a rural area and $120M in an urban area. To calculate the avoided costs based on a life extension for 10 years, these replacement costs were depreciated (straight-line) over a term of 10 years.

Table 4 shows a table for the simple cost-benefit comparison. The cost-benefit table does not include an estimate for reliability and customer perception. In the author’s opinion, these costs are significant but difficult to quantify and justify. The benefit, or avoided costs, for efficiently spending repair/replacement funds depends on the repair method, but currently there is no method for a full pipeline repair that is cheaper than the condition assessment cost. For replacement it can be the same as the life extension benefit case.

Table 4. Simple Cost-Benefit for Condition Assessment of a 10-mile pipeline [16 km] over 10 years

Description	Assumptions	Low Cost*	High Cost**
Costs:			
MFL Condition Assessment	Total project	$1.5M	$2.0M
Steel Repairs	Internal steel patches (4 total)	$30k	$30k
Benefits (Avoided Costs):			
Prevention of a Pipeline Leak	Damage and repair costs (4 leaks total)	$1.2M	$6.0M
Avoided Cost of Depreciation	10 year life extension	$8.8M	$12.0M
	Net Cost Savings:	$8.47M	$15.97M

* Rural Area
** Urban Area

CONCLUSION

Condition assessment projects for large diameter pipelines can be very expensive and operationally complicated. However, over the long-term, the costs of failure, unreliable service, extended life of a pipeline, and efficient repair spending (reduced corrective maintenance and repairs) are greater than the initial condition assessment costs and efforts for the Water Authority.

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Condition Assessment Methods for 1920s Lock-Bar Steel Pipe

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Abstract
This paper presents a case study for the examination of one of the City of San Diego's oldest and most critical water transmission pipelines, a 36-inch (900 mm), 7-mile (11.3 km) long lock-bar steel pipe. A history of the original installation and subsequent maintenance improvements (addition of cathodic protection and re-lining with cement mortar) is discussed, and the results of recent field examinations are presented, along with conclusions. The paper also discusses a number of condition assessment methods and techniques that were utilized on this pipe, including internal video inspection and leak detection; external broadband electromagnetic scanning, guided wave testing, and ultrasonic thickness testing; corrosion testing; transient monitoring; and assessment of valves and appurtenances. Additionally, the paper briefly covers the methodology and results of a risk analysis that was conducted for prioritizing potentially required repair and rehabilitation improvements. The finding of this case study is of direct benefit to other agencies that own and maintain this unique type of steel pipe.

ORIGINAL DESIGN AND INSTALLATION (1926 - 1935)

The City of San Diego's El Capitan Pipeline is one of the oldest and most critical transmission mains in the City's water distribution system. This 36-inch (900 mm), 12.5-mile (20.1 km) pipeline was designed in 1926 and constructed in 1935 to convey raw water from the City's El Capitan Reservoir in Lakeside, California, to, at that time, the City's only water treatment plant, located in what is now the University Heights (mid-city) area. Over the years, new sources of water were obtained from both local and imported supplies, additional treatment plants were built, and portions of the El Capitan Pipeline were either converted to potable water delivery or leased to neighboring water agencies. The 7-mile (11.3 km) portion that was converted to potable water delivery, now called the El Capitan Potable Water Pipeline, is the subject of this paper.

Figure 1 shows an aerial map of the 36-inch El Capitan Pipeline, along with photos of unique features along the pipeline alignment.

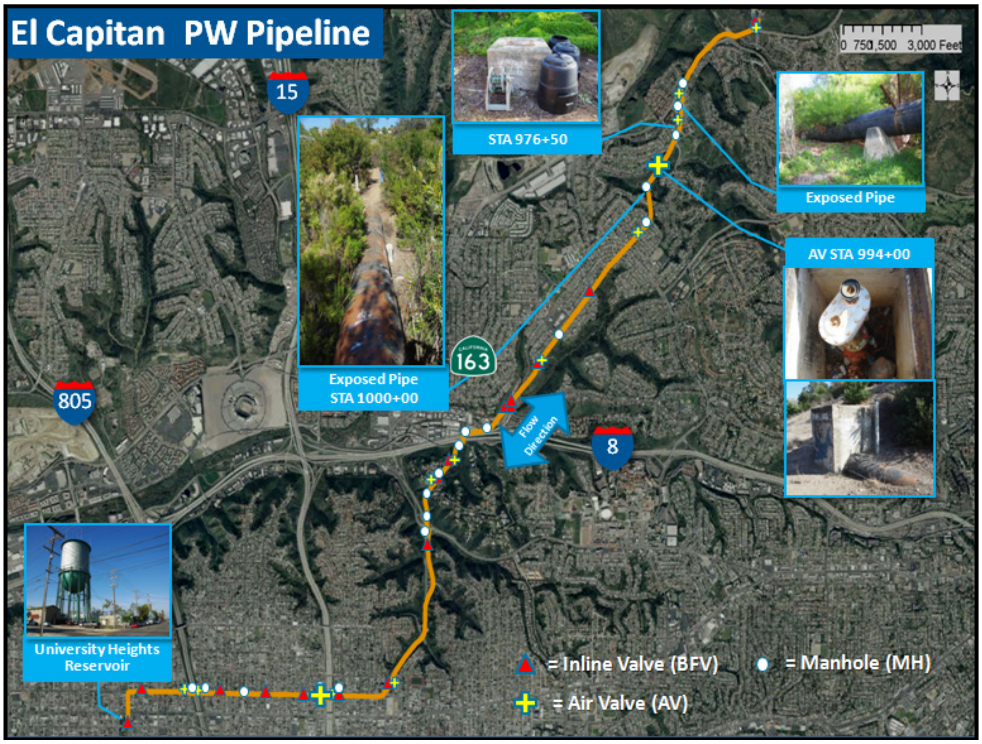


Figure 1. Alignment of El Capitan Pipeline and notable features along the alignment.

HISTORY OF LOCK-BAR PIPE

Lock-bar pipe was invented in Australia in the late 1800s because a cross-county pressure pipeline was needed to convey water from Perth, on the western coast of Australia, to a gold mine near the village of Coolgardie, approximately 350 miles (560 km) from the coast. The most commonly used technology at that time was riveted steel pipe. However, due to the length of the pipe and the pressure required to convey water for such a long length, there was a concern that excessive leakage of riveted joints would be an issue. Welded steel pipe technology was also available, but the cost of welding was significant. The invention of lock-bar pipe addressed the leakage issue of riveted joints and also was less costly than welded steel pipe. The Perth-Coolgardie transmission main was successfully completed in 1905. The pipeline served the goldmine for over 50 years and the small village of Coolgardie grew to a prosperous town with a population of 50,000.

Following its first and major application in Australia, the technology was exported to South Africa and found its way to the eastern shores of the United States via England. The lock-bar pipe technology was later introduced into the west coast of the United States and finally reached the City of San Diego in the 1920s. By that time, the product had experienced significant improvements in terms of its manufacturing as well as quality of joints.

Lock-bar pipe was made of two 30-foot long (9.1 m) steel half-cylinders locked together longitudinally by a hydraulic press edges, under a force of 350 tons per lineal foot, using two H-shaped bars. Figure 2 shows lock-bar pipe details. Figure 3 contains actual photos of lock-bar pipe fabrication and installation.

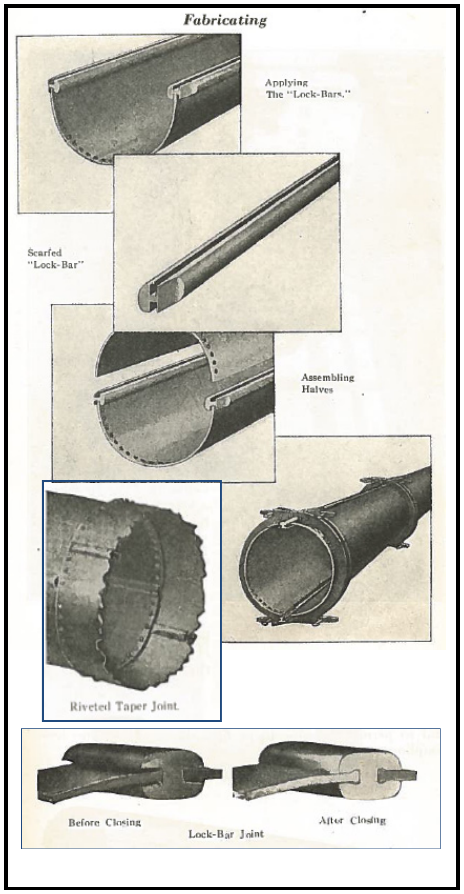


Figure 3. Lock-bar pipe details.

Figure 2.
A. Fabrication of lock-bar steel pipe.
B. Laying of lock-bar steel pipe.

EL CAPITAN PIPELINE

The El Capitan Pipeline is a 7-mile (11.3 km) long, 36-inch (900 mm) diameter lock-bar steel pipe. It has a 50-100 mils thick coal tar enamel coating that was applied at the factory (see photo in Figure 4). The original lining was also coal tar enamel, but a 1/4" (6 mm) thick cement mortar lining was added by mechanical application sometime in the 1950s as a rehabilitation measure. The joints connecting the pipe sections are riveted and may also be welded (see photo in Figure 5). In either case, the joints are electrically continuous. The wall thickness of the pipe was designed to vary along the alignment, in accordance with varying operating and surge pressures and external loading.

The pipeline has an impressed current cathodic protection (CP) system that was installed in 1942, seven years after the pipeline was constructed. The system consists of eight CP stations, each consisting of an anode bed and a rectifier that energizes the anode bed. The City of San Diego routinely tests the CP system and adjusts rectifier output to ensure the pipe is adequately protected against corrosion. Operation and maintenance records of the pipeline are limited and do not indicate any major problems with the pipeline, such as failures or leaks.

The majority of the pipeline was installed using conventional cut and cover construction methods; however, there are eight locations where the original pipe is above ground and supported by the ground surface and/or concrete trestles. These locations offered easy access for visual inspections of the pipeline prior to planning and executing additional field testing and inspections needed for a thorough condition assessment of the pipeline.

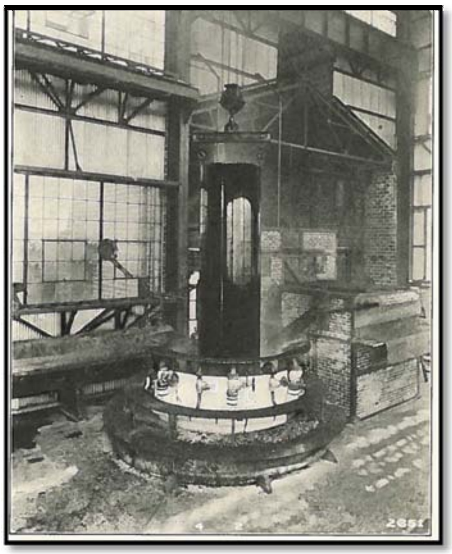


Figure 4. Pipe was vertically dipped in a coal tar bath for corrosion protection.

Figure 5. Exposed sections of El Capitan Pipeline lock-bars, riveted joint, and coal tar enamel coating.

CONDITION ASSESSMENT (2013-2014)

A variety of inspection methods and technologies were used to assess the condition of this 80-year old pipeline. This section contains a description of the methods and an interpretation of the field test results.

External Examination and Basic Measurements

At all exposed reaches and at four excavations, the pipe and coating were visually examined, and ultrasonic thickness (UT) and pit depth measurements were taken (see Figure 6, photos A&B). The following observations were made:

- The coal tar enamel coating was worn out on exposed reaches, and there was corrosion activity under the coating. However, the coating that was remaining was in good condition (see Figure 6, Photos C, D and E).
- When the pipeline was cleaned for guided wave testing (discussed below), some localized pitting was observed and measured, but, in general, the external surface of the steel cylinder was in very good condition (see Figure 6, Photo F). The deepest observable pit was 27% of the wall thickness.

- The steel cylinder was in very good condition at excavated locations. Below are photos of a typical excavated location showing 1) the condition of the coating before removal, and 2) the condition of the bare steel pipe after the coating was removed (see Figure 6, Photos G and H). There was evidence of some minor corrosion, but overall the pipe was in good condition.
- Most of the concrete supports were in poor condition and need to be rehabilitated (see Figure 6, Photo I).
- Wall losses of 5% to 17% of nominal wall thickness were recorded by ultrasonic thickness measurements. However, there was variability in the measurements, and the wall loss is relatively low, considering that the pipe is 80 years old.

Figure 6. Photos of external examination and pit depth measurements.

Corrosivity Study

Soil corrosivity testing consisted of Wenner 4-Pin soil resistivity testing, testing of the cathodic protection system, laboratory analysis of soil samples of native and imported backfill material at the four excavations, and a stray current evaluation.

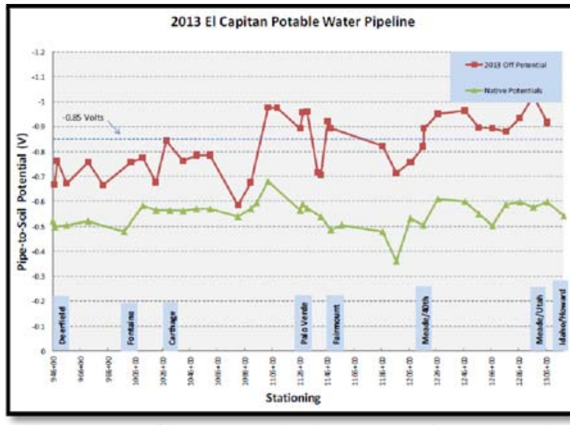


Figure 7. Graphs of pipe-to-soil potential, by station (red = instant off potential, green = native potential).

The results of the soil resistivity tests in conjunction with the performance testing of the cathodic protection system helped guide the locations selected for excavation and visual examination and testing of buried pipe.

The following conclusions were drawn from the corrosivity study:

- Clean (high resistivity) imported backfill material is helping to protect the pipe from corrosion.
- A well-maintained and well-functioning cathodic protection system is protecting this pipeline from significant corrosion.

Guided Wave Testing

Guided wave testing (GWT) was conducted at three exposed pipe locations. In GWT, a collar of guided wave transducers is strapped on the pipe. The transducers must have direct contact with the pipe wall; hence, the coal tar enamel coating had to be completely removed at the GWT locations. The transducers introduce low frequency ultrasonic guided waves that travel axially along the pipe in either direction.

Figure 8. Guided wave testing on exposed reach near a riveted joint.

When the guided waves encounter changes in the cross-section or stiffness of the pipe (produced by welds, supports, corrosion, or other anomalies), reflections occur that propagate back to the transducer collar. The reflections identify areas of potential degradation that may require additional investigation. Typically, guided wave testing can cover 100-150 feet (30-45 meters) along the pipeline in either direction from the transducer collar, and the wave can also be transmitted through pipe that is buried. Figure 8 shows the guided wave testing in action.

Figure 9 shows the sample graphical results from the guided wave scan, which the technician can observe a few minutes after the scan is made.

Although the guided wave testing successfully indicated anomalies along the sections scanned, the ultrasonic wave was not able to pass through the riveted joints, so the use of this technology was limited and was not used as extensively as originally planned.

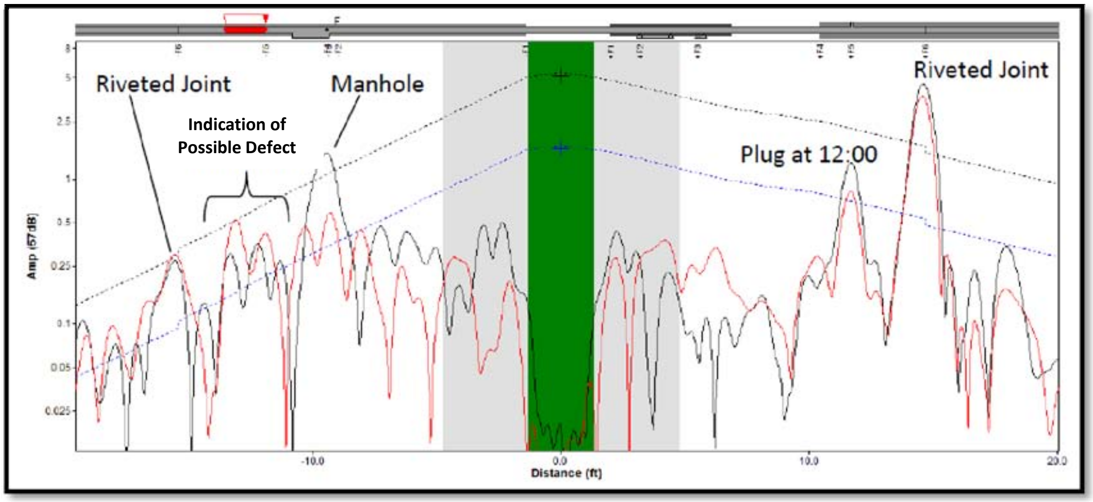


Figure 9. Sample graphical results of GWT.

Broadband Electromagnetic Scanning

Broadband electromagnetic (BEM) testing was conducted at the same three locations where GWT was performed. BEM testing uses a scan of electromagnetic waves to produce a thickness profile of metallic pipe. The thickness profile is used to detect possible corrosion. The data is obtained using a handheld tool. The operator establishes a circumferential and longitudinal grid of 2-inch (5 cm) squares on the pipe, then moves the antenna around the grid, taking readings which are stored on a computer.

Data gathered from the field is processed to provide a "contour map" of the wall thickness (see Figure 10). The results are described as a percentage of the overall volume of material over the scanned area. The areas with apparent wall loss are identified, but the data does not indicate if it is internal or external corrosion. A cluster of pits will show up as a general wall thinning rather than a cluster of pits.

Figure 10. BEM scanning on exposed pipe section.

The results of the BEM testing were consistent with wall thickness measurements taken through UT testing.

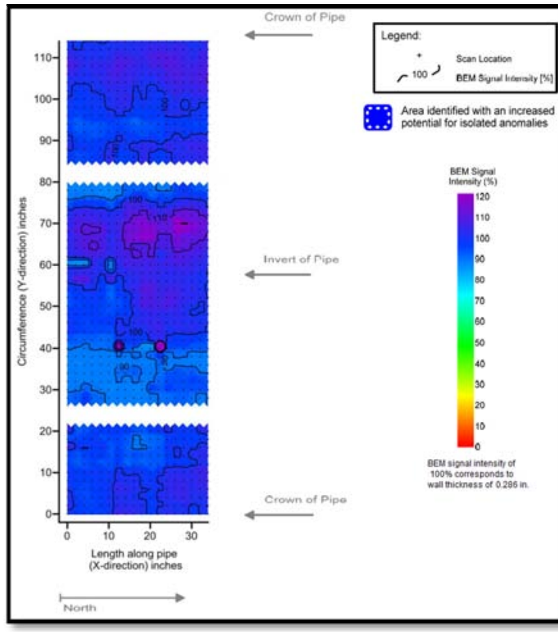


Figure 11. BEM scan contour map of wall condition at this location (STA 971+00).

Internal Video and Leak Detection
The inspection team saw great value in being able to inspect the pipeline interior, particularly the condition of the mortar lining and joints. However, shutting down this pipeline would have been difficult and costly for the City of San Diego. Therefore, the LDS1000™ system by Wachs Water Services was selected to inspect the interior of the pipeline while the line remained in service. This system also has leak detection capabilities, which was an additional advantage of this technology. The LDS1000™ system consists of a tethered cable with an attached sensor head containing a camera, LED lighting, and a hydrophone for leak detection. The sensor head is pulled through the pipeline by a drogue or "parachute" that is propelled by the flow of water. The sensor head was inserted into the pipeline through 2-inch (50 mm) air valve piping that was modified for this internal inspection. City of San Diego O&M staff were on-hand during the inspections to adjust flows in the pipeline to allow the maximum distance to be covered by the tethered camera and hydrophone. While the tethered cable can cover up to 3,000 feet (900 meters), the actual inspection lengths are limited by other factors such as flow, pressure, and bends in the pipe. For this project, the LDS1000™ was used at four locations on the pipeline and covered a total distance of about 5,800 feet (1,768 meters). Figure 12 contains photos of the sensor head and drogue and field set-up.

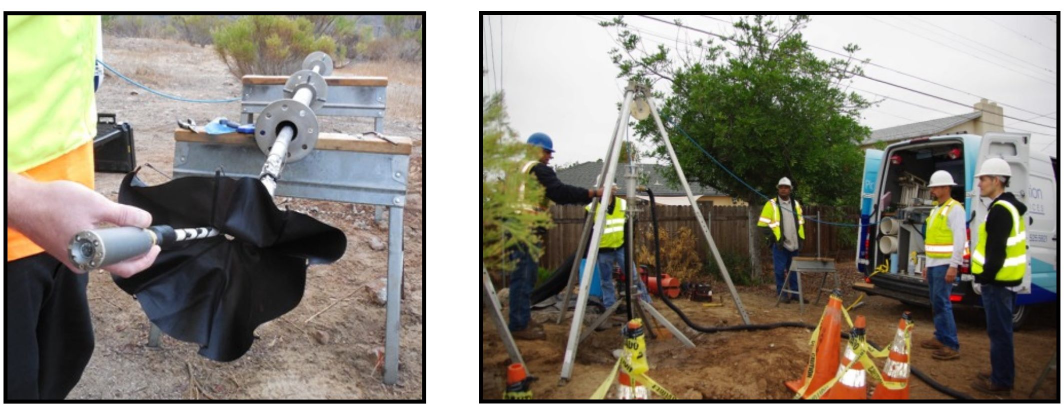
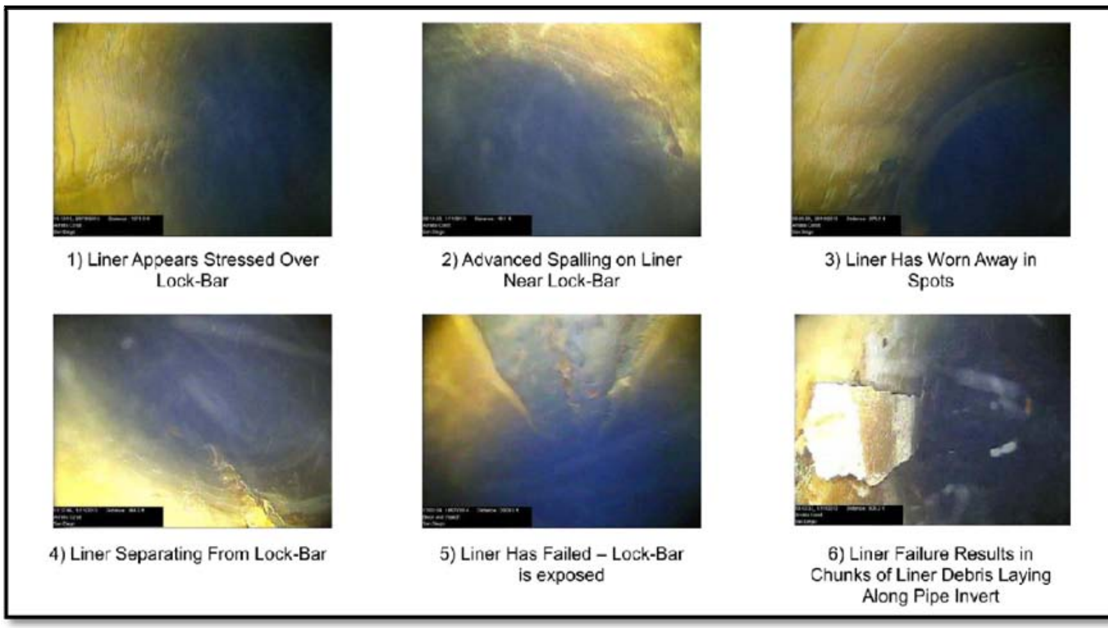


Figure 12. Left: Sensor head and drogue.
Right: General setup for inspection.

Results:

- No leaks were detected in the four reaches inspected.
- There were many minor defects and a few major defects in the lining of the inspected sections. The major defects consisted of large sections of mortar lining falling off the pipe interior, starting at the lock-bar joint.
- There were no joint defects observed in the inspected sections, other than some minor hairline cracks in the mortar lining at the joint.
- Based on the video inspection, the following still images in Figure 13 collectively illustrate the life cycle of liner failure in lock-bar pipe.



1) Liner Appears Stressed Over Lock-Bar
2) Advanced Spalling on Liner Near Lock-Bar
3) Liner Has Worn Away in Spots
4) Liner Separating From Lock-Bar
5) Liner Has Failed – Lock-Bar is exposed
6) Liner Failure Results in Chunks of Liner Debris Laying Along Pipe Invert

Figure 13. Six stages of lock-bar pipe liner failure.

Valve Assessments

A comprehensive condition assessment of a transmission pipeline would not be complete without an inspection and assessment of in-line valves, side-line valves, and pipeline appurtenances (air valves and blow-offs). A valve assessment program, including exercising valves, documenting their location and condition, and repairing inoperable valves, establishes (or re-establishes) system control for distribution operators, which reduces the consequence of a pipeline failure. Wachs Water Services provided valve assessment, evaluation, documentation, and minor repair services as part of the condition assessment of the El Capitan Pipeline. A total of 96 valves were assessed – 47 in-line valves and 49 smaller side-line or appurtenant valves. Figure 14 contains photos taken during the valve assessments.

Figure 14. Left: Exercising an in-line valve with the help of City crews. Right: Uncovering a paved-over-in-line valve.

Results:

- The majority of the valves were found to be in good condition.
- Frozen (i.e., stuck) valves, when encountered, were made operable.
- Buried valves, where located, were uncovered.
- 22 City map discrepancies (for valve location, type, or size) were recorded and corrected.
- 19 recommendations were made for work orders to follow up on needed repairs or to follow up on valves that could not be located.

Transient Monitoring

Like most agencies, the City of San Diego does not have the infrastructure in place to monitor their transmission pipelines for transient pressures. Transient (or surge) pressures can lead to pipeline failure over time.

As part of this condition assessment, internal pressures were monitored at two locations on the pipeline over a 3-month period using the Syrinix TransientMinder, a device that monitors for, detects, and records the occurrences of pressure transients (surges). The locations chosen typically receive the highest pressure fluctuations, as indicated by the City's hydraulic model. Figure 15 shows the installation of a transient monitoring device at one of the two locations chosen, along with a graph of the pressures recorded over the 3-month period at both installation locations.

Results:

- The monitoring program was successful. Diurnal pressure patterns were clearly recorded, but no major transient spikes occurred on this pipeline over the 3-month monitoring period.

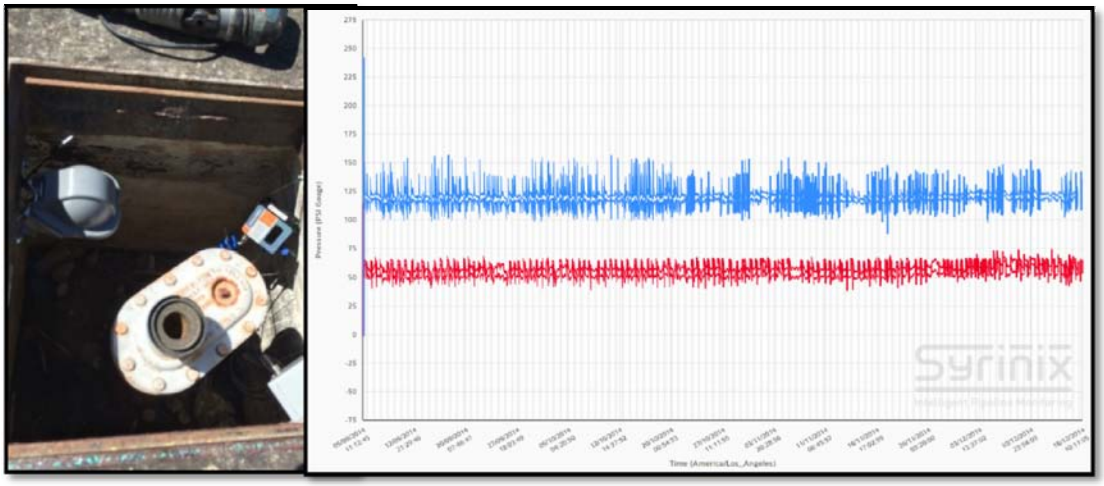


Figure 15. Left: Transient monitoring device installed on air valve piping. Right: Graph of pressures recorded over a 3-month period.

RISK ANALYSIS

A risk analysis was conducted as part of the condition assessment project. Twenty (20) Likelihood of Failure factors and eleven (11) Consequence of Failure factors were developed and applied to this pipeline. The resulting Risk Profile is shown on Figure16. The majority of the pipeline is Medium to Low Risk, with some sections categorized as High Risk.

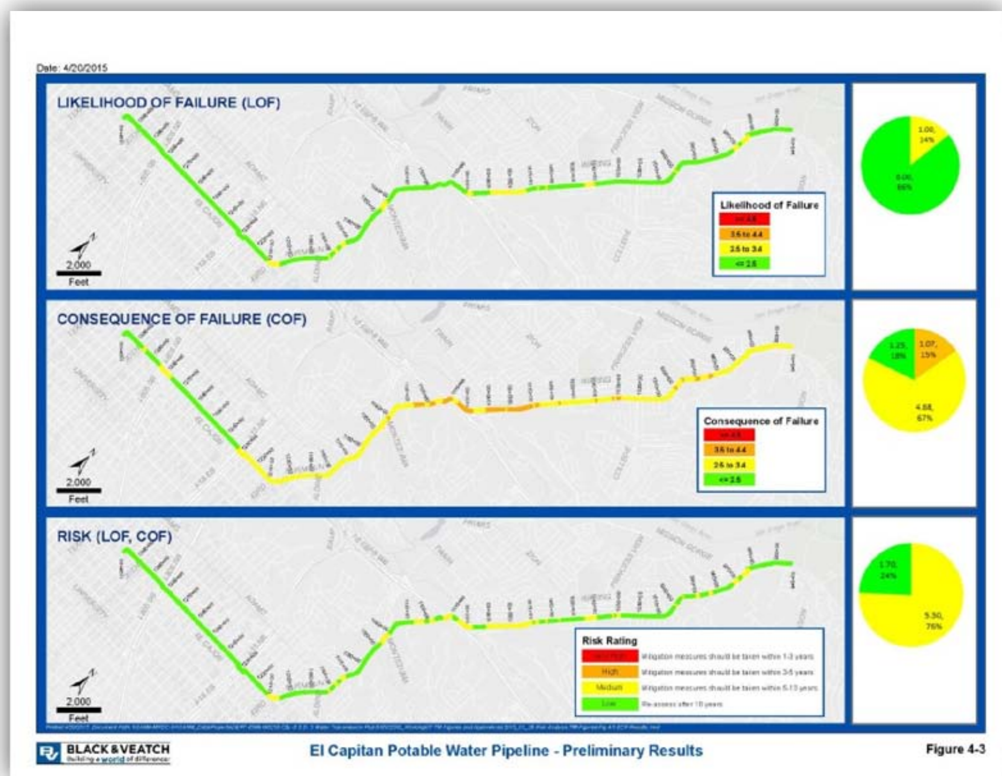


Figure 16. Risk Profile.

CONCLUSIONS

1. The 36-inch (900 mm) lock-bar steel pipe was designed with a high margin of safety. The condition of the pipe pointed to good construction practices utilized during the installation of the pipe.
2. Wall loss, as measured by BEM scanning and UT measurements, ranged from 5% to 17% of the nominal wall thickness. The highest wall loss is occurring on exposed sections of pipe where the coating has failed. Buried sections of pipe appear to be in better condition, as the coating was more intact.
3. In general, the coal tar enamel coating has protected the pipe well. Where it has deteriorated, the pipe wall shows some pitting but not excessive deterioration, most likely because the pipe has a well maintained and well-functioning (impressed current) cathodic protection system.
4. The original lock-bar pipe had a coal tar enamel lining applied at the factory (the pipe was hot-dipped in coal tar enamel). Cement mortar lining subsequently applied in the field does not adhere well to the lock-bars protruding from the interior pipe walls. The lining initially begins to crack and fall off at the lock-bars and then this spreads to more of the lining.
5. With adequate cathodic protection (i.e., continued monitoring and maintenance of the CP system), addressing issues related to internal cement mortar lining, and barring third party damage, this pipe could potentially last at least another 50 years, extending its total service life to 130 years.
6. The City of San Diego is currently evaluating various options for addressing the deteriorating condition of the internal cement mortar lining.
7. A valve assessment program can reduce the consequence of a pipeline failure and is therefore a valuable assessment to conduct on any transmission pipeline, regardless of the extent of other field inspections.

NEXT STEPS

The next step is to conduct an engineering evaluation of the field inspection results, which will also incorporate the results of the risk analysis. The outcome will be the generation of a planning list of rehabilitation and repair improvements for the El Capitan Pipeline, along with associated costs for individual projects and recommended timeframes for completion. The City of San Diego will incorporate the identified projects into their overall capital improvement program.

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A Look Back: Analyzing the Results of LWC's PCCP Condition Assessment Pilot Projects

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Abstract

In 2009, a 60-inch (1500-mm) prestressed concrete cylinder pipe (PCCP) transmission main carrying potable water failed. This break and the hundreds of thousands of dollars in damages inflicted was the impetus for Louisville Water Company's (LWC) PCCP Condition Assessment Program. The program covering 105 miles (169 km) of PCCP pipe was approved as part of LWC's 2010 Capital Improvement Budget and contained two pilot projects that were selected. A 5.2-mile (8.4-km) section of 48-inch (1200-mm) pipe and an 11.5-mile (18.5-km) section of 60-inch (1500-mm) pipe were selected as the two pilot projects based on their perceived criticality, lack of redundancy, damage-causing capability, and the availability of electromagnetic inspection technology. These two projects were inspected with a variety of electromagnetic inspection platforms, high definition (HD) video, and acoustic leak detection technologies. The inspections identified several pipe sections that required a variety of testing, replacement, and rehabilitation options. LWC utilized multiple contractors to employ a variety of repair methods including external steel bands, external post-tensioning tendons, full joint replacement, internal carbon fiber-reinforced polymer (CFRP) linings, and internal hybrid fiber-reinforced polymer (FRP) linings. With the completion of both pilot projects, the paper will discuss all of the issues encountered, items that worked well and those that did not, a comparison of the multiple structural repair methods, data storage issues, LWC's future implementation of the program, and how the program best serves LWC's customers going forward.

INTRODUCTION

In May 2009, a 60-inch (1500-mm) PCCP pipeline failed and poured millions of gallons of water into the neighboring creek and development causing property and facility damages in the hundreds of thousands of dollars. Up to that point, LWC had not experienced a catastrophic failure on a PCCP main. LWC personnel were tasked with creating a condition assessment program to address the 105 miles (169 km) of PCCP pipelines in LWC's transmission system. A 10-year $18.6 million program was presented and approved by the LWC Board of Water Works for implementation starting with the 2010 Capital Improvement Plan. (Williams 2012)

The 105 miles (169 km) were divided into 27 distinct segments, evaluated for priority, and grouped into like budget years to equalize the projected expenditures throughout the program. Two of the highest priority projects were selected and implemented as the pilot projects. The following tables list the specifics of each pilot project.

Westport Rd 48" (1200 mm) PCCP	Distance:	27,597 ft (8,411 m)	
Inspected by: Pure Technologies	Pipe Sections:	1,473	
Inspected on: 11/29/10 - 12/05/10			
Platform(s) Utilized: PureRobotics^{TM}			
Risk of Failure Analysis Performed by: Simpson, Gumpertz & Heger			
Results: Pipes with Wire Breaks:		4	0.3%
(All Classified as Repair Priority 1A or 2A)			
Pipes with No Wire Breaks		1,469	99.7%
(No Leak Detection Performed)			

Figure 1 – Pilot Project #1 Breakdown (Pure 2011) (SGH 2011)

BE Payne 60" (1500 mm) PCCP	Distance:	58,929 ft (17,961 m)	
Inspected by: Pure Technologies	Pipe Sections:	3,081	
Inspected on: 04/18/11 - 04/20/11, 09/22/11, 9/27/11, 10/26/11			
Platform(s) Utilized: SmartBall®, PureCrawler®, PipeDiver®, & PipeScanner®			
Risk of Failure Analysis Performed by: Simpson, Gumpertz & Heger			
Results: Pipes with Wire Breaks:		97	3.1%
(9 Pipe Sections Classified as Repair Priority 1A or 2A)			
Pipes with No Wire Breaks		2,984	96.9%
Leaks Detected		7	

Figure 2 – Pilot Project #2 Breakdown (Pure 2012) (SGH 2013)

The two projects are the primary transmission mains into LWC's eastern service area. The smaller transmission and distribution mains in this area are not capable of supplying the system demands without these mains. The following figure illustrates the pilot projects in relation to each other. The section of 60-inch (1500-mm) main southeast of the connection point is a LWC-designated critical main and would cause a large area-wide boil water advisory should it fail. LWC is in the process of installing redundant transmission mains to eliminate this vulnerability.

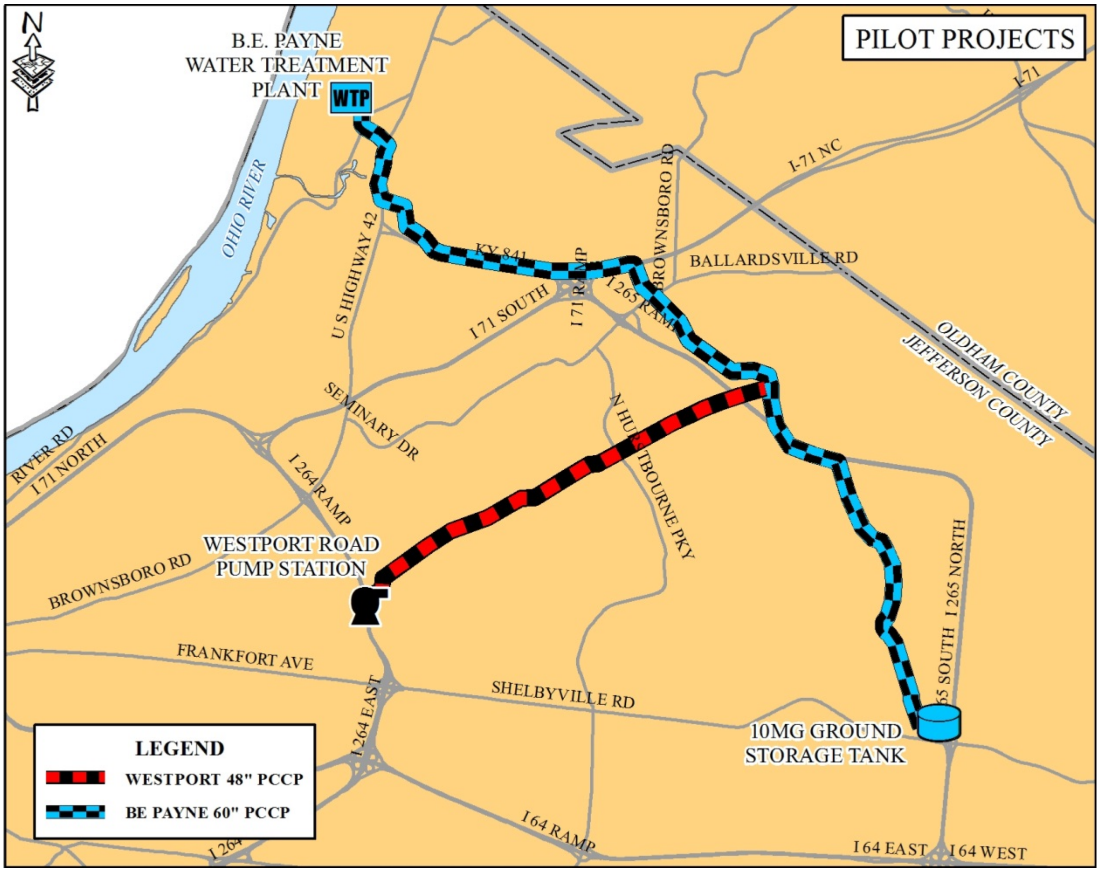

Figure 3 – Original Pilot Projects

It is LWC's practice to rehabilitate, repair, and/or replace pipe sections identified in Repair Priority 1 or 2. LWC's consulting structural engineers calculate the repair priority following the inspection. In some instances, distressed pipe sections not classified as Repair Priority 1 or 2 are addressed due to concerns with the location of the distress on the individual pipe section, the location of the distress along the pipeline, and the impact on adjacent utilities and/or transportation corridors.

PIPE SECTION REPAIR METHODS

After an extensive evaluation of the different repair methods, LWC settled on the following: reinforcing steel repair bands, external post-tensioning tendons (EPT), internal carbon fiber reinforced polymer lining (CFRP), and pipe section replacement. The choice and application of each repair method is contingent upon the pipe size, location along the pipeline, ease of access, whether the pipeline can be taken out of service, and the extent of the damage indicated by the inspection results. All of these methods were utilized during the repair cycle of the pilot projects. In several instances, multiple vendors were utilized for comparison of methods, costs, and quality of workmanship. A comparison of each method and how and when LWC chooses to apply each is described below.

COMPARISON OF LWC'S STRUCTURAL REPAIR METHODS

Reinforcing Structural Steel Repair Bands

Within LWC, there are varying opinions as to the effectiveness of employing reinforcing structural steel repair bands to repair a damaged pipe section. Some view bands as a temporary Band-Aid while others consider it a permanent repair. The thickness of the bands are calculated based on the pipeline's working pressure and fabricated to snugly fit the outer mortar pipeline coating. Any voids in the mortar coating must be repaired prior to installation of the bands. Once the bands are installed, the connecting plates must be welded together to complete the process. LWC backfills the excavation with flowable low-strength concrete fill to protect the bands from corrosion.

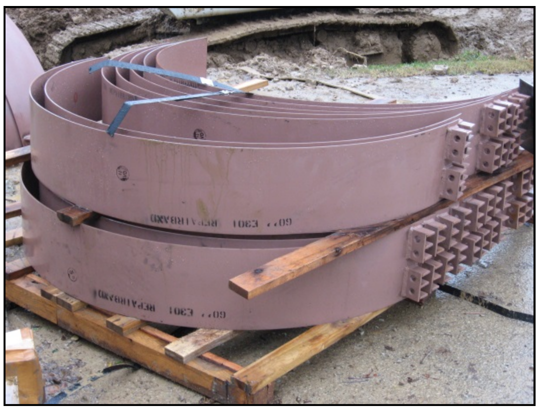

Figure 4 – Repair Bands

Figure 5 – Installed Repair Bands

There are concerns with the effectiveness of the band repair method. If the corrosion of the PCCP prestressing wires has migrated down to the steel cylinder, there is a chance the watertight barrier may be compromised in the future. In addition, the weight of the flowable fill introduces a new strain on the pipeline at the joints of the repaired pipe section and may cause issues in the future.

LWC has chosen to employ the reinforcing structural steel repair bands on pipe sections where 1) the watertight barrier (steel cylinder) has not been compromised, 2) the pipeline cannot be taken out of service or the pressure reduced for repairs, 3) a quick design and repair turn-around is required, and 4) the outer diameter of the pipe section is known.

External Post-Tensioning Tendons

EPT has become the most popular method of rehabilitation at LWC for PCCP pipe sizes greater than 24-inch (600-mm) in diameter. LWC employs a consulting structural engineer to design the EPT system and calculate the necessary tension required. The prestressing of the tendons replaces the prestressing lost with the wire breaks. Once the tendons are installed, the entire system is coated in shotcrete. To protect the system and encourage the shotcrete to cure at a faster rate, LWC has chosen to cover the rehabilitated pipe section with a double wrapping of polyethylene encasement.

Figure 6 – EPT Tendons Being Installed

Figure 7 – Shotcrete Installation on EPT

LWC has chosen to employ EPT on pipe sections where 1) the watertight barrier (steel cylinder) has not been compromised, 2) the pipeline can be taken out of service or the pressure temporarily reduced for repairs, 3) turn-around around time for repairs isn't an issue as EPT system needs to be designed and fabricated, and 4) the exterior mortar coating is sound and intact. In the event, the exterior mortar coating has been compromised, LWC has the contractor chip away the loose mortar, cut and remove any damaged wires in the exposed area and then repair with an epoxy mortar grout prior to post-tensioning.

Based on recommendations from LWC's structural engineer, EPT is not used on pipes 24-inch (600-mm) and smaller in diameter. There are concerns that the EPT could crush the pipe barrel on pipes this small. In addition, LWC has found that it is more cost efficient to replace the pipe section at this size.

<u>Internal Carbon Fiber Reinforced Polymer Lining</u>

CFRP has been selectively used at LWC to repair the larger, greater than 36-inches (900-mm) in diameter, transmission mains from the inside. To ensure proper adherence to the internal pipe column, LWC has instituted a strict set of specifications and quality control procedures for the proper installation of the CFRP. Prior to implementation of these specifications, LWC had several manufacturers and installers review the specifications for completeness and its applicability to pipelines.

LWC employs a consulting structural engineer to design the CFRP system and calculate the proper epoxy resin, the number of carbon fiber layers, and to perform the necessary quality control testing during installation. Once the CFRP has been installed, a new independent watertight pipe section has been generated inside the existing deteriorating pipe section.

All of LWC's CFRP pipeline installations have occurred within the last 5 years. During the next pipeline inspection cycle, each of the CFRP installations will be visually inspected with samples taken for further structural testing. As an interesting side note, the prestressing wires can continue to corrode and not impact the strength of the rehabilitated pipe section. This is one of the benefits of this repair method. During any future inspections, the electromagnetic technology can continue to track the wire break progression without fear of pipe rupture.

Figure 8 – CFRP Impregnating Machine

Figure 9 –Installed CFRP Liner

Due to the highly selective and costly nature of this repair method, LWC has chosen to employ CFRP on pipe sections where 1) pipelines are greater than or equal to 36-inches (900-mm) in diameter, 2) the pipeline can be taken out of service, 3) turn-around time for repairs isn't an issue as CFRP system needs to be designed, installed, and cured, 4) accessibility to excavate pipe section is difficult and costly, and 5) the political capital required to excavate a pipeline is too great.

Pipe Section Replacement

In some instances, a pipe section is damaged beyond repair and requires replacement. All PCCP pipe sections 24-inches (600-mm) in diameter or smaller are replaced if the distress is shown to be in the Repair Priority 1 or 2 by the structural engineer's risk of failure analysis. On larger pipe sizes, the damage to the pipe section must be widespread, have a possibility of negatively impacting adjacent pipe sections, causing the pipe to be out-of-round, or would be cheaper to replace than rehabilitate.

Figure 10 – Wide Spread Corrosion

Figure 11 –Pipe Section Being Replaced

As the replacement of a pipe section is a highly intrusive operation that requires excavation, dewatering, replacement, filling, hyperchlorinating, flushing, and testing, the decision to do so is not taken lightly. LWC performs due diligence in determining the appropriate repair and/or replacement method to address distressed pipe sections.

As LWC was in the pilot project phase with these two projects, it provided the project managers latitude with application of the aforementioned repair methods, many of which had never been utilized at LWC for pipeline repairs. Throughout these projects, issues were encountered that impacted the planning and implementation of future program projects.

ISSUES ENCOUNTERED DURING INSPECTIONS

Several issues were encountered during the inspection of these two projects. A few of them are listed below along with their impact's significance on future projects.

<u>Westport 48" (1200-mm) PCCP</u>

- Isolation of pipelines utilizing in-line and connecting valves.
  Several valves required for the isolation of the pipeline had not been operated in many years and were either not accessible or required minor repairs to be useful. To combat this issue on future projects, a list of all valves required for a pipeline shut down is compiled and field checked 4-6 weeks in advance of the project so any issues can be addressed prior to inspection.
- Air locks during depressurized inspections.
  As part of LWC's condition assessment projects, all air valves and drains are rehabilitated to renew the functionality of each main's appurtenances. Even with LWC's best efforts, there are instances in some pipelines where the as-builts don't quite reflect field conditions. If the site contractor is working on replacement of inoperable air valves during a depressurized inspection, there is the potential of releasing an unknown air lock and endangering persons or equipment in the pipeline. As this did occur

during a robotic inspection, LWC adjusted future inspection runs to halt work on main appurtenances during inspection runs.

<u>BE Payne 60” (1500-mm) PCCP</u>

- Valve operators not following closure plan.

  Prior to a free-swimming acoustic leak inspection, the project manager went over a very specific valve closure plan with LWC’s field operations supervisor. As this pipeline directly supplied a 10 million gallon storage tank, the dispersal of flows into the system instead of filling the tank was a priority. During the inspection run, the valve crews decided to deviate from the plan and “help” by completing the entire valve closure plan before the second valve was to be closed. This “help” caused the tank to fill prematurely and the pumps shut down stranding the free-swimming acoustic leak inspection tool® in the pipeline. Thankfully, the tool was recovered intact, but this instance highlighted an issue that the valve operators, in addition to their supervisor, must be knowledgeable of the plan and the reasoning behind their actions.
- Maintaining constant rate of flow during pressurized inspection.

  Maintaining a constant rate of flow was necessary to provide a consistent rate of inspection. This allows LWC’s inspection company to anticipate arrival times and track the tool throughout the inspection. In addition, a constant flow rate allows LWC to control velocities and the speed at which a receiving storage tank fills. As LWC had issues with the premature filling of a storage tank during the initial free-swimming acoustic leak inspection, a process was created to notify Production Operations to keep storage tanks as low as possible prior to an inspection run and base load pumps to provide a constant rate of flow. New pumps are only added with consultation of the onsite project manager.
- Leaking access manholes during refilling.

  A few of the access manholes that were utilized for depressurized inspection leaked during the refilling of the pipeline. As these manholes were not always at a high point, the filling of the main stopped and, in some instances, had to be partially drained to reopen and reseat the manhole lids. Contractors have been directed to pay special attention when cleaning and resealing the access manholes on future projects.
- Angle of in-line butterfly valve discs during pressurized inspection.

  LWC did not have any issues with the free-swimming electromagnetic inspection tool getting stuck on a butterfly valve during the pressurized inspection. The inspection company informed LWC that if the free-swimming tool did get stuck, the only way to retrieve it would be to excavate the pipeline and physically remove it from the pipeline. As this pipeline had no redundancy, taking the pipeline out of service to retrieve the tool was not an option. Extreme care was taken to operate and leave the valve discs at a pre-determined angle to facilitate the passing of the free-swimming tool.

ISSUES ENCOUNTERED DURING REPAIRS

There were not many issues encountered during the repairs of these two projects. A few of them are listed below.

Westport 48" (1200-mm) PCCP

- EPT.

  The subcontractor performing the installation of the EPT took a while getting the first installation completed. This initial delay concerned LWC but was soon dispelled as the remaining EPT repairs were quickly completed.

BE Payne 60" (1500 mm) PCCP

- CFRP.

  There were a few delays during the preparation and installation of multiple CFRP sections by LWC's subcontractor. Several thousand feet of pipeline were dewatered and dehumidified during this time. The extended period of dehumidification caused the mortar at some pipe joints to crack and spall requiring remediation prior to refilling. LWC had intended to have the subcontractor install a new hybrid FRP system, but issues with the robotic installation equipment caused the subcontractor to substitute CFRP to complete the work.
- Access to Drains.

  Due to unforeseen circumstances, the primary drain needed to properly empty the main prior to multiple CFRP installations was buried and inaccessible. Fortunately, the neighboring property owner allowed LWC to install a new drain and the remaining water was pumped out of the pipeline by the contractor.

ITEMS THAT WORKED WELL

LWC does not take the initiation of new programs lightly. A lot of background research and planning was performed prior to any work being completed. Communication between LWC, the inspection company, the site contractor, and its subcontractors was key to the success of these projects. In many cases, the items being performed were the first time several entities had performed them. The large diameter free-swimming electromagnetic inspection tool pressurized insertion tubes were built specifically for the BE Payne 60" (1500 mm) PCCP Project. All entities were learning as we were going. Without constant honest communication, these projects could have ended badly.

ITEMS THAT DIDN'T WORK WELL

The biggest item that LWC had issues with was the underestimation of the time necessary to set up some of the equipment. The schedules provided did not include enough set up time and caused many of those involved long hours in order to have equipment ready at the designated launch times. LWC has been in contact with those involved to improve the realistic nature of the schedules on future projects.

DATA STORAGE ISSUES

Following the implementation of these projects, LWC has been left with an abundance of inspection data, HD video, and reports. LWC anticipates reinspecting each of the program's pipeline segments every 10 years. What is the best way to compile and organize the available data for future project managers? After several meetings, decisions were made as to the future of the data. LWC's IT department created a shared online folder that could be accessed by multiple LWC project managers, inspectors, managers, etc. The primary data would reside in this master folder. Any high definition video would reside on the DVD's or external hard drives provided by the inspection company until the IT department could decide on the best method of permanent retention. The amount of HD video provided measures in the terabytes and is too big to copy onto a network server.

FUTURE PROGRAM IMPLEMENTATION

LWC's PCCP Condition Assessment Program continues to move forward. LWC has standardized its inspection of PCCP pipelines on one inspection company's technology platforms. Several master agreements have also been created with multiple contractors for site work, pipe replacements, EPT, and CFRP.

Following the two pilot projects, LWC performed a re-evaluation of the program's costs and methodologies. Those methodologies were presented earlier in this paper. The LWC Board of Water Works has continued to approve the implementation of this program. Each of the subsequent projects has found distressed pipe sections. These pipe sections have been either replaced or rehabilitated. Pipe sections that showed minimal distress and weren't categorized as Repair Priority 1 or 2 are being monitored and will be re-evaluated at the 10-year inspection interval.

CONCLUSIONS

The results of this program have provided LWC management with the ability to show that not all pieces of a pipeline are distressed. Performing an inspection and condition assessment of a pipeline allows LWC to rehabilitate the damaged pipe sections prior to a catastrophic failure. These failures not only cause significant system distress but invite unwanted negative publicity and exorbitant repair costs and unwanted property damages. To keep LWC's ratepayers informed as to the significance of this program, LWC has embarked on a proactive public relations campaign to inform the public of what LWC is doing and why we are doing it.

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And the Kitchen Sink—Using a Full Toolbox to Assess a Critical Bulk Water Asset in South Africa

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Abstract

The Department of Water and Sanitation is the custodian of South Africa's water resources and is responsible for meeting the country's current and future water needs. In 2014, the Department of Water and Sanitation undertook a comprehensive inspection and subsequent risk assessment of two of their most critical pipelines. The pipelines total 90 kilometers in length and are comprised of Non-Cylinder Prestressed Concrete Pipe. The assessment included specification development, material testing, hydraulic steady-state and transient analysis, a corrosivity survey, a flown lidar survey, leak and gas pocket detection surveys, electromagnetic inspection, structural analysis, engineering assessment, and risk evaluation. As a result of the assessment, the risk of failure for each of the over 14,000 individual pipes that comprise the transmission mains was quantified and reported to DWS in a geospatial asset management platform.

INTRODUCTION AND PROJECT BACKGROUND

The Rietspruit-Davel and Davel-Kriel Raw Water Transmission Mains span a total of 90 kilometers near Pretoria, South Africa. These transmission mains connect raw water reservoirs located in the towns of Rietspruit, Davel, and Kriel, and are jointly termed the Rietspruit-Davel-Kriel (RDK) Transmission Mains. The RDK Transmission Mains are owned and operated by The Department of Water and Sanitation (DWS) and form a strategic link in a hydraulic scheme that supplies water to several coal-fired power stations. These power stations produce approximately 25% of South Africa's power supply and a reliable water supply is crucial to ensure continued operation. In addition, the transmission mains serve as the only water supply to several small municipalities.

The transmission mains are comprised of Non-Cylinder Prestressed Concrete Pipe (PCP). Please note that this paper abbreviates this type of pipe as PCP, but NCP (Non-Cylinder Concrete Pipe) is seen in other work. PCP is similar in composition to Prestressed Concrete Cylinder Pipe (PCCP), with the exception that the steel cylinder is replaced by longitudinal prestressing wires. The PCP that comprises the RDK Transmission Mains features concrete spigot construction and a steel bell ring, as shown in Figure 1.

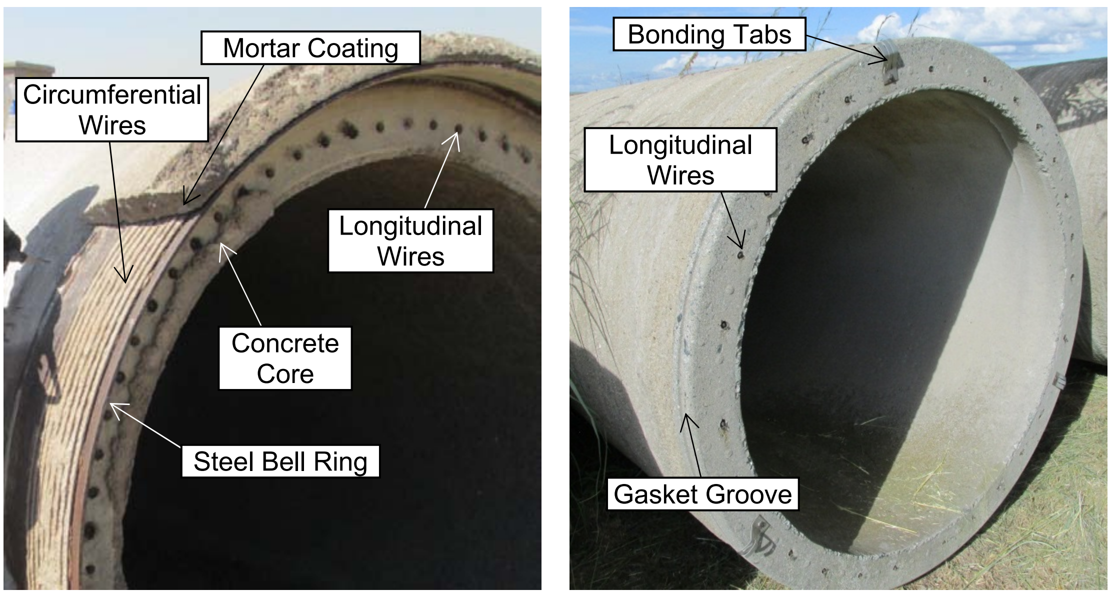

Figure 1. Left – Construction of PCP at Bell. Right – Construction of PCP at Spigot.

The transmission mains were constructed in the 1970s by Interpace Corporation. Both transmission mains have experienced multiple failures in the past that were caused by a range of mechanisms.

The criticality and failure history prompted an investigation into the reliability of the RDK Transmission Mains. The primary objective was to determine the baseline condition of the pipelines, and thus an electromagnetic inspection was required. Electromagnetic inspection is an internationally accepted method for the evaluation of the condition of prestressed concrete pipe. However, DWS also desired a holistic view of the transmission main in addition to the electromagnetic inspection. This included assessments and surveys that aimed to infer some of the root causes of distress, which may assist in slowing future deterioration. The full scope included the following:

- Pipe Specification Development
- Material Testing
- Steady-State and Transient Hydraulic Assessment
- Soil Corrosivity Assessment
- Flown Lidar Survey
- Leak and Gas Pocket Detection Surveys
- Electromagnetic Inspection
- Structural Analysis
- Engineering Assessment
- Risk Evaluation

ASSESSMENT RESULTS

Pipe Specification Development and Material Testing

Specifications were not available for the PCPs that comprised the RDK Transmission Mains. However, as-built drawings indicated that the transmission mains were comprised of seven (7) different PCP classes. DWS made spare PCPs that survived from the original production available for specification development. PCPs pieces were also found along the pipeline route that originated from work completed on the pipeline or previous failures. Detailed measurements were taken of all available pipes to create pipe specifications and drawings.

In addition, due to the history of the pipe manufacturer, extensive material testing was completed to determine the integrity of the pipe constituents and provide inferences on the pipe manufacturing practices. Table 1 summarizes the material testing completed.

Table 1. Material Testing.

Material	Property/Test
Concrete Core	Compressive Strength
Prestressing Wire (both circumferential and longitudinal)	Tensile Strength, Torsional Ductility, Hydrogen Embrittlement Sensitivity
Prestressing Wire (circumferential only)	Residual Wrapping Stress
Mortar Coating	Petrographic Examination, Chloride Concentration, Absorption

The concrete core and mortar coating testing indicated consistent and good quality material. The prestressing wire testing found that the circumferential wires were subjected to the effects of dynamic strain aging and susceptible to hydrogen embrittlement. The results of the longitudinal prestressing wire testing were wholly inconsistent: some wires displayed effects of dynamic strain aging, while other wires did not. The results of all prestressing wire testing indicated that the wire properties likely vary between pipes and even on the same pipe.

Pipeline Surveys

Hydraulic Assessment

A steady-state and transient hydraulic assessment for both the RDK Transmission Mains was completed in March 2014. The hydraulic assessment entailed creating a model that accurately mimics the steady-state and transient behavior of the pipelines under varying operational conditions. The model was calibrated against measured pressure and flow data captured on site. It was found that the model produced conservatively realistic results, which is ideal.

The model outputted the minimum and maximum pressure envelope experienced by both pipelines during standard operating procedures. The assessment found that DWS' current operating procedure does not expose either pipeline to frequent or significant transient pressures. However, specific operational changes were identified to pose a risk of generating severe pressure surges. This information is valuable to DWS when planning operating strategies. An example of the pressure envelope determined through the hydraulic assessment is shown in Figure 2.

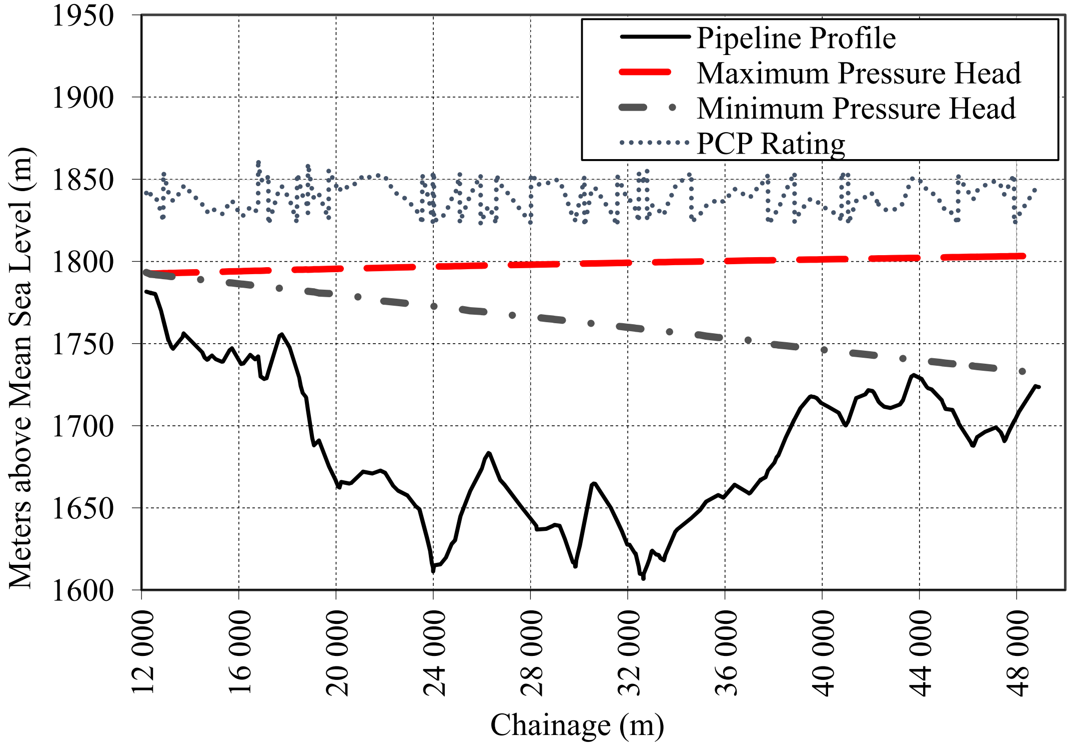

Figure 2. Pressure Envelope for Rietspruit-Davel Transmission Main

Cathodic Protection, Soil Corrosivity, and Pipe-to-Soil Potential Surveys

A local corrosion engineer was subcontracted to complete a cathodic protection audit, a soil corrosivity survey, and pipe-to-soil potential assessment. This work was completed to determine if a correlation existed between the pipeline environment and distress identified during the electromagnetic inspection.

Shown in Figure 1, the pipes that comprise the RDK Transmission Mains were designed to be electrically continuous. However, the assessment indicated that the pipeline is no longer electrically continuous and that cathodic protection is not feasible. This should not be a concern for DWS as impressed current cathodic protection of wire sensitive to hydrogen embrittlement has been observed to be a catalyst to wire breaks.

The soil resistivity testing found a wide distribution of values that indicated a range of severely corrosive to non-corrosive soils. However, soil sampling did not indicate corrosive soils. No correlation was observed between the electromagnetic inspection results and the soil corrosivity survey.

The pipe-to-soil potential assessment found indications of exposed steel at 87% of the 63 test locations. Further, fluctuating pipe-to-soil potential measurements indicated that stray currents might be a problem at four (4) locations. However, no correlation was observed between the electromagnetic inspection results and the pipe-to-soil measurements or the stray currents.

Flown Lidar Survey

In order to obtain high-resolution imagery and assess the condition of the pipeline right-of-way, a flown lidar survey was completed by a local subcontractor. The survey required a plane equipped with global positioning and inertial mapping equipment as well as a lidar scanner and digital camera. The output was a digital terrain model that could be analyzed to determine up-to-date elevation profiles. One typical downfall for employing a flown lidar survey of the pipeline servitude is interference by foliage or above ground structures (e.g., vehicle on a road that lies over the pipeline). However, due to the terrain in the area, this was not an issue for the RDK Transmission Mains.

The vast majority of the pipeline was found to contain the same earth cover as shown in the as-built drawings. This is due to the lack of development in the area and DWS' management of their pipeline right-of-way. One exception however is shown in Figure 3.

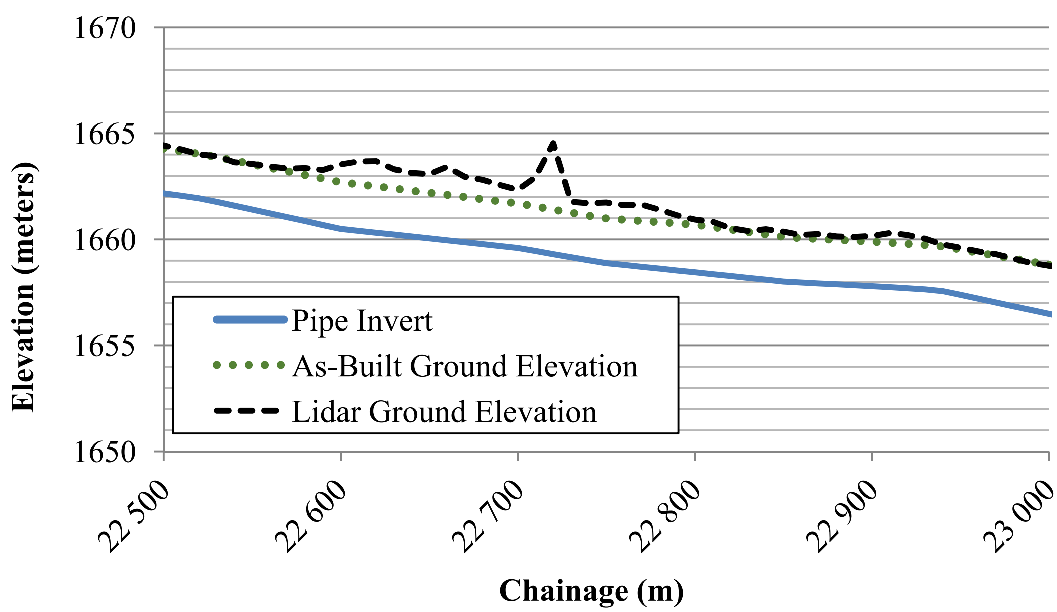

Figure 3. Lidar Survey Results at Location of Excess Earth Cover.

At this location, a contractor placed a pile of dirt from an excavation on top of the pipeline. The lidar survey estimated that the pile was approximately 4-meters high; this information was reported to DWS and the problem was immediately rectified.

Inspection Techniques

Leak and Gas Pocket Detection Survey

A leak and gas pocket detection survey of the entire 90 km pipeline was completed in March 2014 over the course of 5 days. The inspection was completed through use the SmartBall® tool, which is a free-swimming, in-line technology that can detect the acoustic signature of leaks. The survey covered the entire length of the RDK Transmission Main and identified 10 leaks, for an average leak rate of 0.11 leaks per kilometer. Two (2) of the leaks were located at air release valves and were known before the inspection.

Dewatering the RDK Transmission Mains is achieved through 24-inch diameter bottom outlets. Because the inspection tool is untethered and traverses along the pipe invert, it was feared that the tool would fall in the bottom outlets and get stuck. A computational fluid dynamics (CFD) analysis, shown in Figure 4, was performed and found that the probability of the tool successfully passing over the bottom outlet was greater than 95% at a pipe flow velocity of 1.5 m/s (and about 85% at 1.2 m/s).

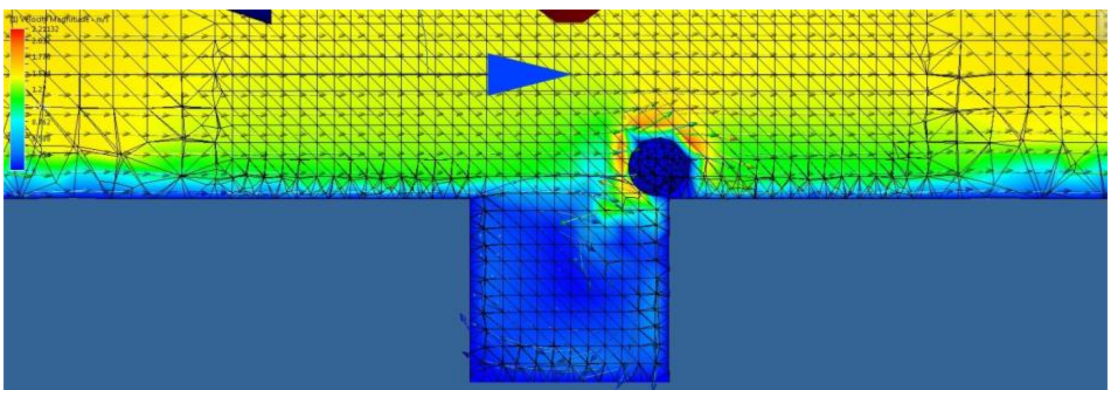

Figure 4. CFD Analysis of SmartBall tool traversing over a bottom outlet.

Sahara® leak detection surveys were subsequently completed at the location of the leaks detected during the initial survey. The Sahara follow-up surveys provided an accurate above-ground leak location and permitted a visual assessment of the leaks from the inside of the pipeline. This provided DWS with more information regarding the leaks, which helped facilitate repair planning.

Electromagnetic Inspection

The electromagnetic inspection was completed through the use of a custom designed PipeDiver® tool. This tool is a free-swimming, in-line technology that is able to detect the electromagnetic signature of broken prestressing wires in prestressed concrete pipes. The electromagnetic inspection was completed in November 2014 and the results are summarized in Table 2 and Figure 5.

Table 2. Summary of Electromagnetic Inspection Results.

Transmission Main	No. of PCPs Inspected	No. of Pipes with No WBs[1] (%[2])	No. of Pipes with WBs[1] (%[2])	No. of Pipes with 0-10 WBs[1] (%[2])	No. of Pipes with 11-20 WBs[1] (%[2])	No. of Pipes with >20 WBs[1] (%[2])
Rietspruit-Davel	5830	5523 (94.7%)	307 (5.3%)	283 (4.9%)	14 (0.2%)	10 (0.2%)
Davel-Kriel	8487	7689 (90.6%)	789 (9.4%)	730 (8.6%)	43 (0.5%)	25 (0.3%)
Combined	14317	13212 (92.3%)	1105 (7.7%)	1013 (7.1%)	57 (0.4%)	35 (0.2%)

1. WBs – Suspected broken wire wraps as detected by the electromagnetic inspection.
2. Percent is calculated as the number of distressed pipes divided by the number of PCPs inspected in each transmission main.

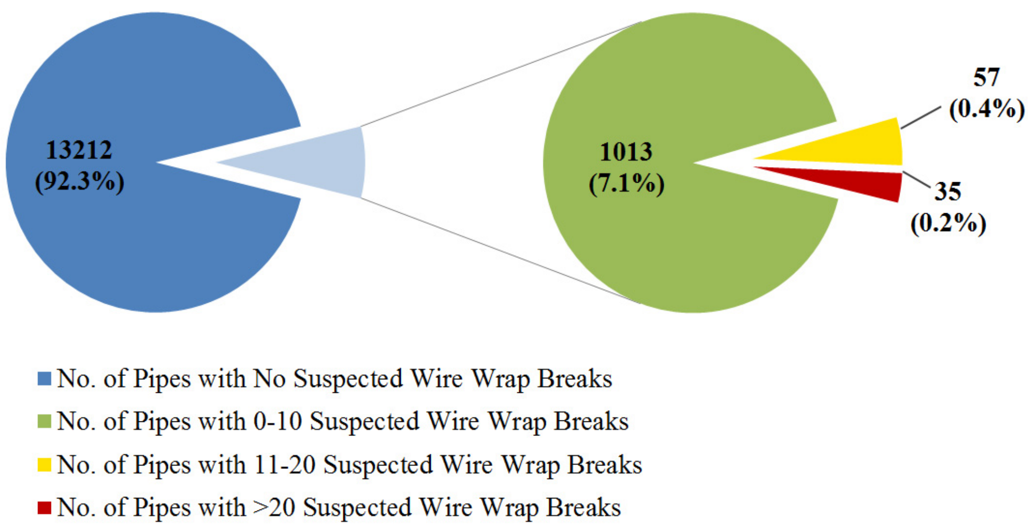

Figure 5. Distribution of Electromagnetic Distress for RDK Transmission Mains.

As shown in Table 2, the inspection found a 5.3% and 9.4% distress rate in the Rietspruit-Davel and Davel-Kriel Transmission Mains, respectively. In both transmission mains however, the majority of the suspected wire breaks was low-level distress (i.e. 5-10 wire wrap breaks). This is favorable for the long-term performance of the RDK Transmission Mains.

Structural Analysis

In order to quantify the structural ramifications of broken prestressing wires in PCP, performance curves were generated for each PCP class in the RDK Transmission Mains. Performance curves quantify the structural consequence of broken prestressing wires for specific PCP designs. A performance curve estimates the condition of a PCP through predefined strain limits at a given number of broken circumferential prestressing wires and internal pressure (Alavinasab et. al, 2011). The pressure used in the structural evaluation was gained from the hydraulic assessment.

RISK ASSESSMENT

The exposure to the risk of pipeline failure, or simply risk, is defined as the product of the Likelihood of Failure (LoF) and Consequence of Failure (CoF) of a pipe and is assigned on a pipe-by-pipe basis. A pipe is considered high-risk if it has both a high LoF and CoF. In this assessment, both the LoF and CoF were ranked on a 1 to 5 scale, where 5 represents a high likelihood or consequence of failure. Ranking systems with too few levels can oversimplify the analysis while a system with too many levels can be too complicated and create an unnecessarily cumbersome analysis. A 1 to 5 scale provides enough granularity to adequately distinguish between different LoF and CoFs levels.

Likelihood of Failure

The LoF ranking system was based on the leak and gas pocket detection surveys, the electromagnetic inspection, and the structural modeling. A LoF rating was assigned to every pipe using an algorithm that considered the following:

- Presence of Leak
- Total Number of Expected Wire Wrap Breaks
- Number of Wire Wrap Break Zones
- Location of Wire Wrap Break Zones
- Number of Wire Wrap Breaks in Each Zone
- Structural Modeling Limit States

The material testing, transient assessment, soil corrosivity survey, and flown lidar survey are not directly included in the LoF rating. These assessments aim to find pipeline conditions that can accelerate deterioration but do not directly assess the integrity of the transmission mains. It is expected that the conditions would manifest as damage detected by the leak detection or electromagnetic inspections.

Consequence of Failure

The CoF ranking system employed a triple-bottom line approach to evaluate the social, environmental, and economic impacts and costs of a pipe failure. The triple-bottom line approach evaluated the categories shown in Table 3.

Table 3. CoF Evaluation Categories.

CoF Category	Type of Cost
Public Health & Safety	Social
Effect on Other Infrastructure	Social
Impact of Discharged Water	Social
Level of Service – Redundancy and Storage	Social
Level of Service – Extent of Outage	Social
Public Image and Regulatory Impact	Social
Environmental Impact	Environmental
Direct Costs	Economic

The relative importance of each CoF category was quantified by a weighting factor.

With the exception of the public image and regulatory impact category, all categories were evaluated with a quantitative scoring rubric. Qualitative scoring rubrics are easier to create, but can be difficult to interpret and the results can vary significantly depending on the personnel completing the analysis. Conversely, once a quantitative scale has received approval from all stakeholders, analysis is straightforward, defensible, and consistent.

Risk Ratings and Recommendations

The product of the CoF and LoF rating was calculated for each pipe and sorted from high to low. This provided DWS with a prioritized list of the pipes of concern. The list was also categorized as extreme risk, high risk, medium risk, and low risk.

RECOMMENDATIONS AND MANAGEMENT

The prioritized list and risk categories provided DWS with two management options that could be customized to their budgetary constraints, as follows:

1. If DWS had a strict limit on the funds available to address pipes of concern, they could simply repair or replace the number of pipes they can afford. Choosing the pipes to rehabilitate would be straightforward and based on the risk rankings.

2. If DWS had more flexibility in the budget, they could choose to repair all of the pipes in the extreme and high risk categories. This is more favorable than the above option for increasing the reliability of the transmission mains.

The results of the risk assessment were presented to DWS in a geospatial asset management reporting platform.

CONCLUSIONS & LESSONS LEARNED

DWS completed a comprehensive inspection and risk assessment to assess the condition of two of their most critical assets. Because all inspections were completed by in-line technologies that do not require pipeline dewatering, there was no impact to the industrial and municipal customers. Due to the length of the transmission mains, full scale replacement is not possible. The recommendations from this condition assessment will extend the useful life of these assets and return the pipelines to a reliable condition.

This project exemplified the importance of accurate record keeping and forensic investigations of failed assets. There have been multiple failures in both pipelines, but no written records were captured to detail the exact cause. Accurate records and forensic investigations would have been able to help focus the assessment. One of the project recommendations was to implement a record keeping system.

This was the consultant's first time employing a lidar survey and the results were positive. These transmission mains were perfect candidates for this type of survey, since they were primarily in undeveloped areas with minimal foliage. Lidar surveys may only be possible in undeveloped areas or areas where the utility diligently maintains their right of way; however, these are the pipelines that are least likely to have issues with overloading.

The LoF rating was evaluated on an "or" basis, meaning that a number of different failure modes were scored and the highest rating was assumed to the pipe's LoF. The CoF rating was evaluated on an "and" basis, meaning a number of failure costs were evaluated and the CoF rating was the sum of all costs. This scenario is intuitive, since it can be assumed that a failed pipe will have a single failure mode, but its failure will have multiple costs. These types of rating scales are recommended for future projects.

At the time of the submittal of this paper, DWS is planning pipe replacements and validations.

REFERENCE

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Large Diameter Pipeline Asset Management for Sustaining Silicon Valley's Water Needs

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Abstract
The Santa Clara Valley Water District (District) manages an integrated water resources system that includes the supply of clean, safe water to local water providers who deliver drinking water to homes and businesses in the heart of Silicon Valley. This paper describes the District's large diameter pipeline management and rehabilitation strategy, including corrosion control, and provides some examples of what we've learned over the past 10 years. The District manages a 142 mile large pipeline infrastructure. The first pipelines were constructed in the 1950's. These initial pipelines were reinforced concrete pipe and steel, followed by a major period of Prestressed Concrete Cylinder Pipe (PCCP) construction, followed up in the latter years by coated steel pipe. Beginning in early 2000, concerns regarding PCCP reliability raised the need for establishing a long-term program and strategy for managing the inspection and rehabilitation of all existing pipelines. The first component of the District's strategy is corrosion control. The design lives of early pipelines were usually not more than 50 years. Currently, we are looking at extension of asset life to 100, 150, and possibly 250 years depending on the existing condition of the pipelines. The next component is vigilant semi-annual monitoring and preventative maintenance activities. Thorough in-pipe inspections are scheduled on a 5 to 10 year window depending on pipe condition and are usually coupled with pipeline rehabilitation and repair activities. Pipeline rehabilitation and repair activities incorporate a comprehensive assessment of the current pipeline and appurtenant condition. Based on the condition during rehabilitation, appurtenances are usually replaced, civil, mechanical, control system, and electrical upgrades and modifications to existing structures undertaken, and any internal pipe repairs or other enhancements completed. Over the past 10 years, 70% of the District pipelines have been inspected and undergone rehabilitation.

INTRODUCTION
Santa Clara Valley, located South of San Francisco Bay, became widely known as Silicon Valley in 1970's as electronic and digital technology entered mainstream society. This big regional shift to technology started during World War II, when the Valley was known as the "Valley of Heart's Delight" for its booming agricultural industry. Initially, as agriculture developed, there was a great supply of surface water and groundwater resources, relative to the needs of the agricultural community. By 1900, irrigated orchards began spreading as advancements in well water systems, electric pumping, and related innovations ensued. An exponential increase in water

wells and groundwater withdrawals eventually led to detrimental land subsidence. As groundwater problems intensified, the local population eventually voted for a water conservation district to develop dams and recharge basins. The water conservation district evolved into what is now known as the Santa Clara Valley Water District (District).

Water demands heightened with an increase in urban-industrial development and residential expansion in the 1950's and it was clear that additional water supplies would be needed. In 1965, the state of California began delivering water to the Valley via the 72-inch South Bay Aqueduct and within a few years, 40 years of progressive land subsidence was halted. In 1987, additional water supplies were delivered from the federal Central Valley Project through the San Luis Reservoir and the 96 to 120-inch San Felipe pipeline system.

Today, the District has expanded to include the management of an integrated water resources system that includes the supply of clean, safe water, flood protection and stewardship of streams. The District effectively manages 10 dams and surface water reservoirs, three water treatment plants, an advanced recycled water purification center, a state-of-the-art water quality laboratory, nearly 400 acres of groundwater recharge ponds and more than 275 miles of streams.

DISTIRCT'S PIPELINE INVENTORY

The District provides over 121 billion gallons of water annually to over 1.8 million people, 15 cities, 13 water retailers, 4,700 direct well owners, and hundreds of farmers and ranchers, along with managing a 142 mile large diameter raw and treated water pipeline infrastructure.

Figure 1: District's In-County Distribution System Map

The first District pipelines were installed in 1965 to help deliver imported raw water to the county from the Hetch Hechty pipeline and South Bay Aqueduct. These first pipelines were made of steel and ranged in size from 66 to 78-inches in diameter. This water was used for groundwater recharge, in an effort to replenish the aquifer and halt the ground subsidence that was occurring at that time. The next set of District pipelines were constructed in 1967, and consisted of 30 to 84-inch diameter steel pipe, used to deliver treated water to the community from the District's Rinconada Water Treatment Plant. In 1974, the District completed the construction its Penitencia Water Treatment Plant, and with that came the installation of more steel pipe to help delivery treated water to the community. In the 1980's, the District completed the construction of a third water treatment plant, and installed a good number of PCCP for raw and treated water delivers within the county.

In the 1990's, the District added more steel pipe to its inventory following the construction of an intertie with the San Francisco Public Utilities Commission (SFPUC). The new steel pipeline helped unify regional distribution of treated water between the District and SFPUC customers, for use in time of need.

Table 1: Breakdown of District Owned Pipe Types and Lengths

Material Type	Miles of Pipe
Prestressed Concrete Cylinder Pipe	80
Steel Pipe	57
Other Reinforced Concrete Pipe	5

HISTORY OF DISTRICT CORROSION CONTROL EFFORTS

Corrosion control has long been known as an effective method of protecting and extending the life of pipelines and appurtenances, reducing water pipeline breaks, associated water loss and improving public safety. The Districts corrosion control strategy uses a combination of good bonded coatings coupled with cathodic protection systems.

Bitumen coal tar and leaded paint coatings have been observed on older pipelines constructed in the 1950's. In the 1960's, corrosion test stations were installed as part of pipeline construction projects. These early corrosion control test stations played a role in static monitoring of pipelines, looking for variations that might be interpreted as possible corrosion. The District also began using non-conductive materials (insulating joints) to separate different pipelines into smaller sections, which helped minimize corrosion cells, and began systematically applying various coatings as anti-corrosion measures.

It wasn't until the 1980s that the District began placing large diameter pipelines and tanks under impressed current cathodic protection. At that time, staff had limited knowledge of corrosion and used consultants for cathodic protection design work. However, at that time the corrosion community was still inexperienced in understanding the behavior of larger diameter mortar coated pipelines under cathodic protection. Consultants were unaware of where isolation points were required and this lead to inconsistent rectifier and anode well placement. This was further illustrated by the numerous systems that were over designed with more than double the needed rectifiers. Adding to the dilemma of inexperience was the introduction of pre-stressed concrete cylinder pipe (PCCP), which has narrower potential requirements for cathodic protection as the pre-stressing tendons are susceptible to hydrogen embrittlement by the over application of current that can result in explosive pipeline failures.

Following several years of corrosion program neglect, starting in 2000 when former corrosion staff retired, the program languished without dedicated staff or other resources until 2007. In addition, many of the paper and electronic files from that timeframe were lost as a result of the retirements, computer upgrades, and loss of databases. In 2007, the District hired an experienced corrosion technician to help the pipeline engineering team get the program started up again. The District hired JDH Corrosion Consultants (JDH), in the January 2008 to assist in this effort. JDH focused on the inventory all systems and rehabilitating the existing neglected cathodic protection systems, as well as identifying unprotected assets. Over a three year period most of the District's critical pipelines were reviewed, with the exception of the San Felipe System. Each pipeline segment was analyzed to determine the effectiveness of the cathodic protection system and adjustments and repairs were made to ensure that most of the critical pipeline segments and tanks that had cathodic protection were functioning. Many protected pipelines had unresolved issues and some systems required further adjustment and surveying to equalize their performance after years of neglect. Several aging systems and components were also found to be in need of repair and/or replacement.

The District has continued to use consultants since one corrosion technician was unable to monitor and maintain the entire system. An entry level additional corrosion technician was hired at the beginning of 2012 and following training, he transferred to a higher paying mechanic position after a little more than six months on the job. It was not until mid-2014 that an experienced technician was eventually hired.

As a result the lack of program continuity and succession planning, no significant cathodic protection was added from 2000 through 2010. Since 2010, approximately 20 miles of protection has been added to pipelines and all pipelines are now routinely monitored in accordance with National Association of Corrosion Engineers (NACE) standards. The San Felipe System Conduits, which are managed by the District under an agreement with the United States Bureau of Reclamation, are now cathodically protected, with the exception of the twin pipes crossing the Calaveras fault and a small rectifier that protects a short section of the 120-inch diameter Pacheco Conduit. Once environmental clearances and permits are obtained, these conduits will be protected for the next few years to come.

Today the majority of the District's owned large diameter pipelines are under cathodic protection, with only a few short sections remaining unprotected.

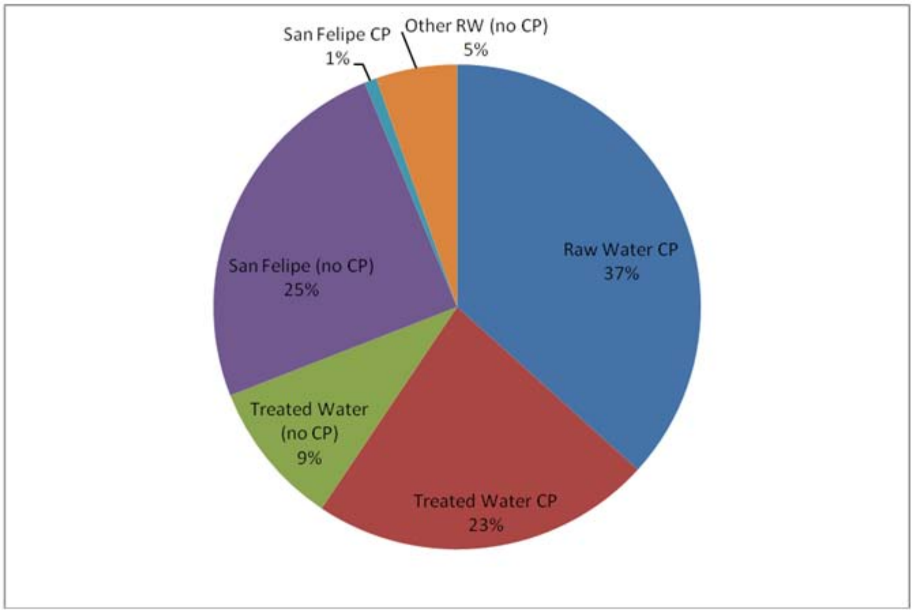

Figure 2: Chart of District Pipelines under Cathodic Protection

To resolve these few sections and to ensure existing cathodic protection systems remain viable, close cooperation with District pipeline mechanics and engineers has been paramount. The involvement of corrosion control technicians and the District's consultant has been critical to successfully protecting pipelines, appurtenances, and tanks from corrosion. Pipeline rehabilitation efforts have also provided corrosion staff with an opportunity to perform internal inspections of the pipeline, measure pipewall thickness, replace failed insulation points, inspect coatings, as well as install test stations critical to cathodic protection.

PIPELINE CONDITION ASSESSMENT AND REHABILITATION PROGRAM

Beginning in the year 2000, the District initially tried to undertake pipeline inspection and rehabilitation projects separately. After several projects were completed, it became obvious that a programmatic approach was needed and that all maintenance work needed to be covered under CEQA and NEPA for federal facilities. This coincided with the beginning of a formalized asset management program in the water utility. District staff developed a 10-year Pipeline Maintenance Program (PMP) and completed an EIR to cover all work performed, which was approved by the District Board in November 2007. The program was the first major comprehensive rehabilitation effort for many of the raw and treated water pipelines since their construction and placement into service as far back as the 1950's. The goal of the program was to reduce the number of unplanned shutdowns and emergency repairs due to severe corrosion of appurtenance connections, which is typical for many of the pipelines constructed over the past decade.

The PMP identifies the range of maintenance activities and provides protocols and procedures for carrying out these activities; including conveyance system inspection, repair, and preventative and corrective maintenance. The PMP identifies the maintenance process, the activities, and defines a wide spectrum of measures and practices to protect the environment. The preventive and remedial maintenance activities associated with the program address current District policies regarding asset management and protection and also accounts for changes in the California Department of Public Health (CDPH) design guidelines (California Water Works Standards), which require the District to upgrade treated water pipeline air valves by adding above ground vent lines in an effort to reduce the potential for entry of polluted flood water into the pipeline.

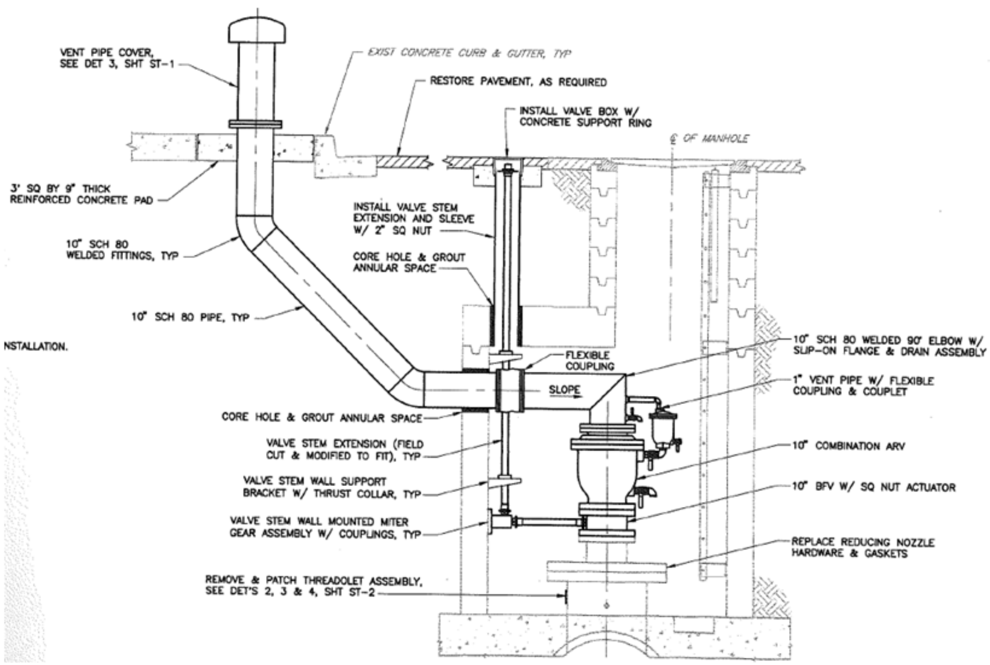

Figure 3: New Venting for Combo Air Release Valve Assembly

Over the past 15 years, the District Pipeline Maintenance Program has successfully completed the rehabilitation of about 100 miles (70%) of large diameter raw and treated water conveyance pipelines. Starting with the initial efforts to inspect and then rehabilitate pipelines beginning in 2000, the scope and complexity of every following project has increased. Changes are constantly incorporated into the program to address lessons learned on earlier projects, so that the program is constantly evolving and striving for efficiency. Pipeline condition assessment, preventative and corrective, maintenance, and rehabilitation efforts are now integrated into District's asset management program. This has allowed the District to prioritize pipeline inspection, rehabilitation, repair, and replacement efforts together with other system and facility shutdowns. It also allows the District to understand the full cost of pipeline maintenance and rehabilitation and what level of investment is needed to properly care for this infrastructure into the future. The incorporation of corrosion control strategies can prolong the life of buried pipeline and vault infrastructure significantly, with the potential for massive savings by deferring replacement, since the pipeline remains in a safe and reliable condition.

The next phase of District's pipeline maintenance program will add additional detailed investigation of vulnerability of our pipelines and pump stations. Elements of these assessments include information on pipeline fragility, system fragility, emergency repair procedures for all pipelines, and the time and cost to return functionality. Identified vulnerable areas will be repaired and strengthened in accordance with our asset management program.

Condition Assessment Program

The District started its formal condition assessment program in the early 2000's, with the use of standardized forms for recording the condition of pipelines and appurtenances. Today, these forms are available on handheld devices used during field inspections. The forms allow for a condition assessment of the pipelines on a rating scale of 1 to 5 (excellent to unserviceable/end of life). The field data is later transferred from the handheld device into the District's computer maintenance management system (CMMS), where it is stored. These ratings become a component of the District's risk score of the pipeline (probability of failure) and help determine maintenance projects for the upcoming years. District is able to inspect each of its facilities once every two years; however, internal inspection of pipelines are conducted once every 5 to 10 years, because these facilities need to be dewatered in order to facilitate inspection.

Individual pipeline condition assessment strategies are dependent on the pipes material. The District's pipelines are primarily pre-stressed concrete cylinder pipe (PCCP) or welded steel pipe (WSP). Because of the ability for PCCP to fail catastrophically, the District diligently monitors the condition of its PCCP through visual internal inspections and the use of eddy current testing to detect wire breaks, which provides a good indication of the of the structural integrity of the pipe. The Districts WSP are often smaller in size, which inhibits the ability for staff to perform visual internal inspections of the pipe. In these cases, the District performs video inspections of the pipe to assess the condition.

The District intends to condition this practice for its condition assessment program and may evaluate emerging technologies that allow for inspecting and monitoring pipelines while the lines are in service. Currently, pipelines must be drained for inspection, and repair work takes place while the pipelines are drained. This work often consists of rehabilitation and repair pipeline sections and appurtenances, such as replacing corroded air release valves, repairing or replacing line valves, repair vaults, and repairing minor leaks with internal pipe joint seals. This maintenance work helps prevent pipeline leaks, and helps keep pipelines in service.

Pipeline Rehabilitation

The District's pipeline rehabilitation efforts have grown over the years, with staff completing 1 to 2 full rehabilitation and inspection projects each year. The scope of the rehabilitation efforts often consists of internal pipeline repairs, pipeline inspection, and the repair or replacement of pipeline appurtenances. In the earlier years, pipeline rehabilitation efforts were often limited to the inspection and replacement of key pipeline appurtenances along the system and the majority of the work during this period were completed under service purchase orders for maintenance services. However, since the District has undertaken the PMP, our pipeline rehabilitation efforts have become much more Capital intensive. This has resulted in projects requiring the preparation of full scale plans and specifications, Engineer's reports, and compliance with local and State contract codes for full scaled bidding and award of contract. 2010 marked the first year under the PMP where pipeline rehabilitation efforts included turnouts and guard valves. Later rehabilitation efforts added flow meter replacement, electrical upgrades, replacement of buried line valves, and the addition of new manholes to improve operation and maintenance flexibility.

As the needs for pipeline rehabilitation projects increase, there has also been increased pressure to complete more work in shorter timeframes. The consequences of a failure to bring a pipeline back according to the shut down schedule are very significant, with potential impacts to water supply and the water retailers providing water to the community. This has resulted in rehabilitation projects requiring more and more resources and expertise, so they can be properly managed from start to finish. Additionally, the requirements imposed by local jurisdictions continue to increase, such as constraints on work hours and traffic requirements, paving moratoriums, and other infrastructure projects undertaken by others that can significantly affect our work windows.

Environmental Compliance

The District is its own lead agency under the California Environmental Quality Act (CEQA), which means that the District certifies, under CEQA, the projects that our agency intends to carry out. The District also utilizes Best Management Practices (BMPs) and Mitigation Measures (MMs) to ensure that projects avoid environmental impacts to the extent feasible given resource constraints. The criticality of BMP/MM deployment varies with the predominant landscape of any given project. One of the main concerns for projects in the more rural portions of our system are issues surrounding impacts to habitat for sensitive amphibians. Most of these facilities convey raw water, where there is usually less concerns surrounding dechlorinating drinking water or disinfection slugs. On the other hand, the more urban portions of our system primarily consist of treated drinking water pipelines that require dechlorination prior to discharging into the storm drains and creeks. The most common BMPs utilized in these projects are the inspection of the waterways upstream and downstream of discharge points to ensure that aquatic species egg masses are not dislodged and washed downstream due to the increased stream flows. In addition, the District also filters some of the water discharged into the creek system to reduce the potential for the introduction of exotic species into the channel.

The sometimes herculean efforts needed to obtain environmental permits and clearances, together with extreme levels of environmental monitoring has added significant cost to rehabilitation projects and requires an increased focus on planning so that pipelines can be taken out of service during increasing limited shutdown windows.

Drought Impacts

The ongoing drought in California is beginning to severely limit and change long term rehabilitation plans. Reduced local rainfall and limited snowpack in the Sierra's has reduced State and Federal water allocations. Our water retailers have been requested to pump more groundwater and utilize other sources of water to service customers. Raw water delivery to many recharge ponds has been terminated or significantly reduced, and surface treated water supply to retailers have been cut 20% so meager surface water supplies can be preserved to provide ongoing flow to the water treatment plants. The uncertainty over water supply is creating a planning nightmare with scheduling large capital projects such as water treatment plant upgrades and seismic retrofit of several of our large dams together with pipeline rehabilitation projects. Shorter pipeline shutdown windows are being demanded and these complex projects are being limited to sometimes 6 weeks or less of a shutdown.. On top of this, engineering and maintenance staff resources have been reduced with some being redirected to projects related to drought response. This year and the next will see the District cut 50% of the planned rehabilitation projects due to resource limitations. The difficulty of taking key raw and treated conveyance pipelines out of service and scheduling work into the future should not be understated and the opportunities to take critical active pipelines out of service for inspection and maintenance needs to be taken when they present themselves. It has been difficult to pass on such opportunities for maintenance due to the drought, other pipeline projects, or limited staff resources, as senior experienced staff caution that deferring major pipeline maintenance activities until another opportunity presents themselves, may in some cases mean deferring maintenance to the point of failure.

DAMAGE AND REPAIR OF A 78-INCH DIAMETER PCCP AT THE ALAMITOS CREEK CROSSING

The District has been lucky that our portfolio of PCCP have been reliable following the initial scare and concern from early inspections in 2000, when evidence indicated possible problems that were subsequently resolved through direct inspection of pre-stressing wires. The District has seen relatively little evidence of degradation of the pre-stressing wires. The District continues to use electro-magnetic surveys for condition assessment and for baseline survey.

While most of the PCCP inspected have been in good condition, the District has encountered some problems. During an inspection on the Almaden Valley Pipeline in 2008, a twenty (20) foot section of seventy-eight (78) inch diameter PCCP was observed to have circumferential cracking of the lining and water intrusion near its crown at the 1:30 clock position, approximately six (6) to eight (8) feet away from the upstream joint near the Alamitos Creek crossing. To stop the water leakage, the pipe received an interim repair using two Weko seals at the crack.

Following the initial repair with the Weko seals, an electromagnetic inspection was performed, in conjunction with an internal visual inspection of the pipeline. There was an anomalous signal in this pipe segment and the data was subsequently analyzed, which indicated that there were 15 wire breaks on the pipeline. Upon further investigation, the cracked pipe section was found to be immediately adjacent to the southernmost abutment of a bridge crossing (Almaden Expressway-Alamitos Creek bridge), which was constructed after the pipeline was installed. The pipe encasement did not extend beneath the footing of the bridge abutment as was shown in the original bridge construction drawings. Moreover, the bridge construction drawings showed a one and a half foot clearance over and on both sides of the pipe. It became clear that the bridge abutment was stressing the PCCP and consulting engineers from Simpson, Gumpertz and Heger performed a preliminary analysis on the pipe section in question during the shutdown and concluded that it was safe to return the pipeline to service. However, they recommended that the District should have the pipe section repaired in a relatively short time period, not in excess of three years. The consultant also noted that the pipeline was at risk of having significant damage if an earthquake were to occur prior to the completion of the repairs on the impacted pipe section (Gumpertz and Heger, 2008).

Because of the poor access to remove or replace the pipe and the unknown level of deterioration of the pipe cylinder, the District decided to perform a full structural in-situ repair by constructing a full ½" epoxy coated welded steel pipe inside the original PCCP. This in- situ repair was completed in Spring of 2011 and required one short two week shut-down, minimal excavation, and minimal environmental permitting.

Figure 4: Internal Repair of 78-inch PCCP with Epoxy Coated Steel Lining

CONCLUSION

Pipeline inspections and maintenance work can be very costly due to complicated work conditions. The work must often be done quickly, as pipeline can only be shutdown for short periods of time. The work often takes place inside the pipe, which requires important safety practices, and many times, field conditions are very different from what is expected.

The economic engine of Silicon Valley must have a resilient water conveyance system operating reliably and capable of resisting earthquake and other hazards. The only way to ensure long-term reliability is to ensure that the condition of pipelines is understood and that condition changes over time. Resilience is the ability to anticipate risk, limit impact, and bounce back rapidly in the face of a turbulent event. The resilience of the District's large diameter conveyance network for raw and treated water is imperative so that the economic miracle of Silicon Valley can continue.

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A Repair Program to Minimize Failure Risk of Highly Distressed PCCP Circulating Water Lines

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Abstract

Arizona Public Service Company's Cholla Power Plant has four units with circulating water lines made of 66 in. to 72 in. diameter prestressed concrete cylinder pipe (PCCP) in Units 2, 3, and 4. PCCP in these three units has been in service for thirty-four to thirty-seven years. Inspections over the years have shown a high level of distress throughout the pipelines due to widespread corrosion of prestressing wires. A condition assessment and repair program has been developed and performed over multiple outages with the goal of minimizing the risk of failure of the pipeline and avoiding unscheduled shut down. This paper presents a unique condition assessment and repair program implemented over the past six years.

INTRODUCTION

The Cholla Power Plant is a four-unit, 995-megawatt coal-fueled power plant in northeastern Arizona. Arizona Public Service Company (APS) owns and operates Units 1, 2, and 3, which are capable of producing 615 megawatts of electricity combined, and also operates the 380-megawatt Unit 4. The circulating water (CW) pipelines in Units 2, 3, and 4 that are the subject of this paper are made of prestressed concrete cylinder pipe (PCCP), embedded cylinder type (ECP), except for a few fiber-reinforced polymer (FRP) pipes, steel special pieces, and risers. A summary of the PCCP in the pipelines is presented in Table 1.

Table 1. Summary of PCCP in APS Cholla CW pipelines[(1)].

	Diameter (in.)	Approx. Length (ft)	Years in Service	Manufacturer	Prestressing Wire Class	Shorting Straps	Approx. No. of PCCP
Unit 2	66	1,700	37	Interpace	IV	No	119
Unit 3	66	2,600	35	Interpace	IV	No	173
Unit 4	72	2,200	34	Ameron	III	Yes	128

[(1)] Reported information is approximate and for intake and discharge lines combined.

In the late 1990s and early 2000s, APS experienced some failures in the CW pipelines and performed repairs including encasing several pipes in concrete, replacing some pipes with FRP pipes, and installing carbon fiber reinforced polymer (CFRP) and glass fiber reinforced polymer (GFRP) liners. One of the discharge lines was lined completely with GFRP. In 2005, APS began using internal electromagnetic (EM) inspection for condition assessment and inspected Unit 2 twice, Unit 3 once, and Unit 4 twice by 2008 using the EM inspection method. These initial EM inspections indicated that 22% of PCCP in Unit 2, 34% of PCCP in Unit 3, and 25% of PCCP in Unit 4 were distressed with broken prestressing wires. Challenged by the high rate of prevalence of distress, APS retained Simpson Gumpertz & Heger Inc. (SGH) in 2008 to provide the engineering services needed to maintain the pipeline at an acceptable risk of failure and minimize shutdown by effectively using the planned outages to inspect the pipelines, perform condition assessment, failure risk analysis and repair prioritization of distressed pipes, and repair pipes with high risk of failure. Since 2008, APS has had five outages resulting in repair of about one hundred segments of PCCP to date.

CONDITION ASSESSMENT

Condition assessment of the CW lines consisted of an initial investigation in 2008 while the units were in service and subsequent studies in planned outages since then. The initial investigation included external inspection of selected pipes using wire continuity testing to verify the results of previously performed EM inspections (Zarghamee et al. 2012), laboratory tests on soil and pipe mortar coating samples for chloride ion profile, structural evaluation of pipe design classes according to the current AWWA Standard C304, and development of failure risk curves for distressed pipes to determine their repair priorities at the time of inspection and in the future.. During subsequent planned outages, condition assessment of the CW pipelines continued to include internal visual and sounding inspections, new EM inspections, correlation of the results of internal and EM and inspections, evaluation of distress growth rate, and repair prioritization that is based not only on the EM results but on a combined evaluation of all inspection results. The following sections discuss how each condition assessment method has been utilized and the experiences gained from their use on the severely distressed CW lines.

Electromagnetic Inspections

The number and location of broken wires in PCCP determined by EM inspection can be used in failure risk analysis as described below to determine how close a distressed pipe is to failure at the maximum internal pressure. Prediction of broken wires involves comparison of EM signals with those obtained from calibration testing of the same or similar pipe.

Prediction of distress is subject to uncertainties in interpretation of signal distortions, and such uncertainties are exceptionally higher for ECP without shorting strap as in Unit 2 and Unit 3 CW lines, while shorting straps in the Unit 4 CW lines are expected to improve the EM distress prediction. This is because a pipe with shorting strap shows a linear relationship between the actual number of broken wires and the distortion of the signal, while a pipe without shorting strap shows a large distortion for a single broken wire and a lower resolution as the number of broken wires increases.

In the case of Cholla CW lines, interpretation of the EM signals and prediction of distress by the Inspection Company and evaluation of results by SGH was more challenging than for typical pipelines, especially for the Unit 2 and Unit 3 lines due to the extent of corrosion. In the 2005-2009 EM inspections, the number and location of broken prestressing wires were predicted; however, subsequent verification by external inspections (see below) revealed that one of the pipes that was thought to be non-distressed was actually severely distressed with all prestressing wires corroded away, and the concrete core cracked longitudinally. A close examination of the EM signals revealed that a non-distressed pipe and a pipe with all prestressing wires corroded away have similar signals except for a shift in the phase of the signal, resulting in initial misinterpretation of data and inaccurate distress predictions. This necessitated reevaluation of all EM data by the inspection company for Unit 2 and Unit 3 CW lines and development of a new distress categorization system without providing number and location of broken wires. In descending order of distress, pipes were classified into Category 1A, 1, 1*, 2, 2*, 3 and 4, where Category 1A, 1, and 1* represent "pipes with high signal phase shift with majority of wires broken", Category 2 and 2* represent "pipes with lower signal phase shift that are likely distressed across the majority of the pipe length but also are likely to have some good wires", Category 3 represents "pipes with moderate distress" for which prediction of the extent of distress is possible, and Category 4 represents "pipes with minimal or no wire breaks." Considering that a majority of pipes in both Unit 2 and Unit 3 were classified in Category 1A, 1, 1*, 2, or 2* and that repairs had to be spread out over multiple outages, repair prioritization was extremely important and was performed by first externally inspecting selected pipes from different distress categories followed by internal visual and sounding inspections, and then using all data collected to evaluate the failure risk and repair priorities of CW lines. It should be noted that technology advancements in recent years may have improved some of the limitations of EM inspections such as those experienced in the earlier inspections at Cholla.

External Inspections

In Cholla, as in many other power plants, external inspections are minimized as much as possible to avoid excavations that could interrupt plant operations and due to various superstructures that limit access to pipes; however, when performed, external inspections provide valuable information. Examples include the following:

- During the initial investigation in 2008 while the pipelines were in service, one pipe in Unit 2, three pipes in Unit 3, and one pipe in Unit 4 that were identified by EM inspection to have 10 to 95 wire breaks were excavated and externally inspected. In addition, one non-distressed pipe in Unit 2 (according to EM) was excavated for reference. Inspections indicated that EM inspection misidentified a pipe as distressed with 15 broken wires although it had no broken wires, misidentified another pipe as non-distressed although it had 24 broken wires, and underestimated the wire breakage in two pipes by 22 and 62 wires, bringing the accuracy of the EM results into question. Some wires, while not broken, were splitting along their length, indicating susceptibility to brittle fracture. One pipe with predicted 95 broken wires was found to have a longitudinal crack with up to 1.5 in. width (Figure 1a) at the crown with fully corroded wires along the entire length of the pipe.

- Both Unit 2 and Unit 3 were re-inspected in the first following outages in 2009 and 2010, respectively, and the number and location of broken wires was re-predicted. Initially, excavations for external inspection were deemed unfeasible for this outage, and pipe repair and replacement decisions were made based on EM results, failure risk analysis, and internal inspections. During excavation of one severely distressed pipe for replacement, the adjacent pipe, which was predicted by EM to be non-distressed and even used as a reference "good pipe" for prediction of distress in other pipes, was found to be fully distressed with all prestressing wires corroded away, mortar coating delaminated, and core cracked longitudinally (Figure 1b). After Inspection Company diagnosed the problem to be related to the interpretation of the EM signals as explained earlier, all results were reanalyzed, and the new distress predictions were presented in terms of distress categories as discussed above. Excavation and wire continuity testing on selected pipes in Unit 2 and Unit 3 showed that four pipes in Category 1 or 1* had 130 to 220 broken wires, three pipes in Category 2 or 2A had 114 to 242 broken wires, one pipe in Category 2B had only seven wires, and one pipe in Category 3 had 13 broke wires, indicating that Categories 1 and 2 are likely both severely distressed, while Category 2B may not be as distressed as originally thought.
- The above mentioned external inspections also provided an opportunity to check the prestressing wire diameter and spacing, mortar coating thickness, type of backfill, and moisture at pipe depth, and take samples for laboratory testing, discussed in the next section.

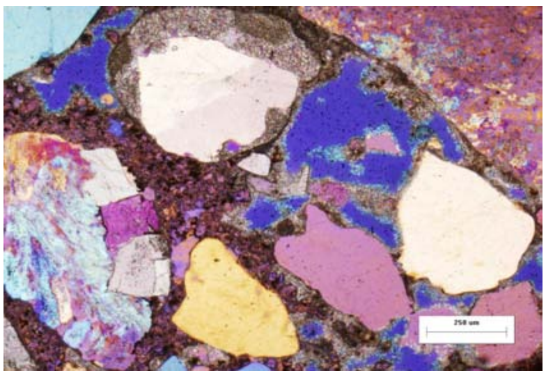
(a) Up to 1.5 in. wide crack along pipe

(b) Wire impressions left on concrete core after wires were fully corroded away

Figure 1. Obervations from external inspections.

Laboratory Testing

Laboratory testing and petrographic inspection performed on samples of soil and mortar coating collected from the Cholla site showed that the environment is highly aggressive and corrosive to PCCP, and the design of the pipeline should have included additional protective measures in form of moisture barrier, silica fume in the cement, or cathodic protection. The chloride content of the soil samples ranged between 660 ppm and 3,800 ppm, well beyond the allowable limit for PCCP of 400 ppm, and chloride ions have permeated through the mortar coating and reached the wires. The corrosion potential has caused hydrolysis, which in turn has resulted in lowering of pH of the mortar coating and acceleration of corrosion. Petrographic analysis showed variable mortar coating quality with void content close to the allowable limits, altered paste surrounding entrapped air due to carbonation, and heavy staining of the paste due to prestressing wire corrosion (Figure 2).



(a) Altered paste surrounding entrapped air voids due to carbonation

(b) Heavy staining of paste due to severe corrosion of steel wire in pipe excavated for replacement

Figure 2. Obervations from laboratory testing of mortar coating.

Internal Visual and Sounding Inspection

Internal visual and sounding inspections have been performed in the CW lines in every planned outage in Cholla since 2009, and the condition of each pipe has been documented with traceable notes and photographs. This allows evaluation of progression of distress between outages and effective repair prioritization of many pipes in the high EM distress categories in absence of predictable number and location of broken wires. For example, there have been cases where repair of a pipe in a lower EM distress category was prioritized over that of a pipe in higher distress category as the former had internal signs of advanced distress and the latter did not.

Internal inspection also allows for evaluation of the condition of previous repairs after exposure to the chemistry and temperature of the circulating water based on visual observations as well as laboratory testing of samples taken from the previous repair materials. For example, the pressure capacity of the existing GFRP liner in one of the CW lines in Cholla was shown by laboratory testing to be lower than the strength required to support the working pressure of the intake line and potentially close to that of the discharge line, not including the transient pressure and safety factor, indicating that such liners may be relying on strength contribution from the host pipe and may not be able to withstand the design loads as the host pipe degrades further. Pipes with such existing liners are also inspected in detail for signs of distress, and in some cases, the GFRP liners were removed and replaced with new CFRP liners.

(a) Large hollow-sounding area in inner core

(b) Longitudinal crack found after removal of existing coating at a suspect location

Figure 3. Typical observations from internal visual and sounding inspections.

STRUCTURAL EVALUATION

Pipes designed for circumferential effects according to the semi empirical design methodologies in effect at the time of their manufacture may not meet the requirements of the current AWWA 304 design procedure that uses a limit states approach, and therefore may be prone to distress. The procedure for design of pipe for hydraulic thrust has also evolved significantly over the years, and pipes in service today, especially if installed in soft soils, may be prone to thrust-related distress such as circumferential cracking, tearing of steel cylinder, joint openings, etc. Consequences of distress resulting from structural inadequacy may range from failure to increased rate of degradation, for example due to direct exposure of prestressing wires or steel cylinder to the environment.

In Cholla, the same classes of pipe are used in the Unit 2 and Unit 3 pipelines, but Unit 3 is operated at higher pressures. Structural evaluation and development of limit state envelope curves using UDP software indicated that the pipe class that is used in a significant majority of the lines satisfied all design limit states in Unit 2 but violated a strength limit state associated with yielding of prestressing wire at springline under transient conditions in the intake line of Unit 3. This means that, when all other factors are the same, Unit 3 pipes are more prone to becoming distressed than other pipes. Structural evaluation for hydraulic thrust was also used in Cholla, where thrust restraint is provided by concrete thrust blocks. For example, a pipe adjacent to a bend was found to have circumferential cracks with corrosion marks, and subsequent thrust restraint analysis indicated that the steel cylinder did not have nearly enough thickness to resist longitudinal stresses if the thrust block at the bend moves. This resulted in repair of this pipe not only for circumferential effects but for thrust as well, and an increased attention to be paid to similar pipes near bends in future outages.

FAILURE RISK ANALYSIS

Failure risk of pipes in Categories 1 and 2 is determined directly from EM signal by the inspection company. For pipes not in Categories 1 and 2, the failure risk of distressed pipes is evaluated using the risk curves, generically shown in Figure 4, which define the relationship between the maximum pressure in the pipe and the effective number of broken wires required to reach serviceability (e.g., onset of coating cracking), damage (e.g., structural cracking of the coating and high stresses in the wires adjacent to broken wire zone), and strength limit states (e.g., rupture), each represented by a separate curve, while accounting for the effect of earth load and pipe and fluid weights. The limit state curves divide the plots into different zones of repair priority, RP1A through RP4, in the order of descending risk of failure and need for repair (Figure 4). Once the number of broken wires is detected by EM inspection, the effective number of broken wires can be calculated by uncertainty analysis and plotted on the risk curves at the maximum expected pressure for the distressed pipe to evaluate the failure risk. The details of the failure risk analysis procedure can be found in Zarghamee et al (2003).

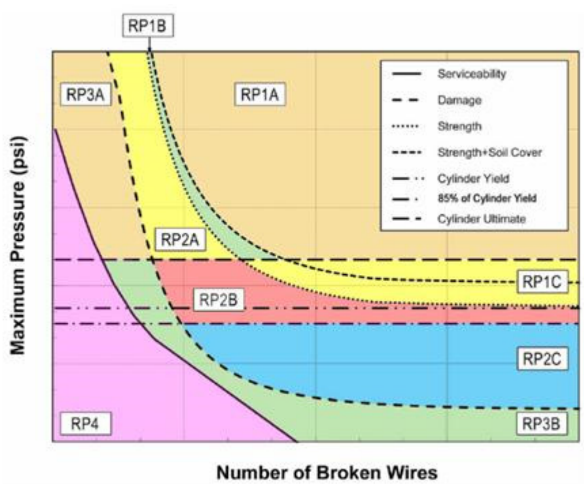


Figure 4. Failure risk curves.

In Cholla, the number and location of broken prestressing wires could not be reliably predicted by EM inspection as discussed above, and instead, the EM inspection company classified pipes into distress categories with definitions such as "majority of wires broken", etc. As a result, typical repair prioritization discussed above could not be used; instead, a new repair prioritization method was developed based on combined evaluation of all information on each pipe including the number of broken wires required for failure based on failure risk curves, EM distress description including "extent" of wire breakage, results of internal inspections (e.g., longitudinal cracks, hollow-sounding areas), comparison of the maximum pressure with the capacity of the steel cylinder alone, whether the pipe was previously repaired or not, and the condition of existing repairs, if any. For example, in the intake line of Unit 3, the maximum working-plus-transient pressure is greater than the ultimate capacity of the steel cylinder; 35 to 40 broken wires are predicted to cause failure of pipe at the maximum pressure; and some distressed pipes have existing GFRP liner while some do not. Typical standard length pipe has about 150 wire wraps, which means that many pipes in EM distress categories 1 and 2, which were reported to have a majority of their wires broken, were in high risk of failure if they did not have an existing liner, or at the mercy of the existing liner. In the discharge line of the same unit, the maximum pressure is equal to the yield capacity but lower than the ultimate capacity of the steel cylinder; 80 to 90 broken wires are predicted to cause failure of pipe at the maximum pressure; almost all pipes are classified in EM distress Category 1; and a GFRP liner with variable condition exists along the entire line. Preliminary laboratory tests show that the GFRP liner in the intake line has adequate pressure capacity for working conditions but not for transient conditions, and the capacity of the GFRP liner in the discharge line is not known but is likely lower due to continuous exposure to higher temperatures. Combined evaluation of such data and selection of a number of pipes for repair based on available resources in each outage is not straightforward. The repair prioritization method developed and used for this unit for the last six years has been to prioritize the repair of highly distressed Category 1 and 2 pipes in the intake line as much as possible even if this meant to repair Category 2 pipes in the intake line before Category 1 pipes in the discharge line, or remove and replace some GFRP liners while there are other distressed pipes without any liners.

REPAIR

The repair program was aimed to minimize the failure risk of the pipeline by repairing distressed pipes at high risk of failure within the constraints of available budget and outage time of unit for maintenance. This requires consideration of different repair methods (e.g., internal lining, external post-tensioning, etc.), relative cost of repair methods, ease of internal and external access to pipes, and other factors. While the external repairs typically have the advantage of lower cost, internal repairs have the advantage of not requiring any excavations.

In Cholla, the preferred method of repair is lining with internally bonded carbon-fiber reinforced polymers (CFRP) (Figure 5a). In some outages where the number of repaired pipes had to be particularly maximized, external repair by post-tensioning and shotcrete encasement (Figure 5b) was also used in parallel with internal CFRP repairs.

CFRP liners are typically designed as a standalone system to resist all internal and external loads without strength contribution from the host pipe, and sometimes designed to act compositely with the inner concrete core of the pipe but without any contribution from the rest of the pipe to reduce the cost of repair. SGH has performed a significant amount of research over the years on CFRP renewal and strengthening of PCCP including experimental and analytical research for the Water Research

Foundation (Zarghamee and Engindeniz, 2014; Engindeniz and Zarghamee, 2014), development of watertightness measures for CFRP liners (Zarghamee and Engindeniz, 2015), characterization of cure behavior of epoxies used in CFRP repairs (Engindeniz et al., 2014), and development of quality assurance procedures for field inspections (Engindeniz et al., 2011). Such work has formed the technical basis of the AWWA Draft Standard for CFRP Renewal and Strengthening of PCCP that is currently in development. CFRP liners used for repairs in Cholla were designed according to the state of the art at the time of repairs, and have performed successfully to date. Typical planned outage duration of about three weeks, which allows repair duration of about two weeks, has allowed APS to repair more than ninety distressed pipes with CFRP liners over the last five outages in different units.

External post-tensioning repair is designed according to AWWA C304 Standard for Design of PCCP and by using the UDP software to consider the combined effects of internal and external loads.


Figure 5. Typical internal CFRP and external post-tensioning repairs.

TYPICAL OUTAGE PLANNING AND EXECUTION

To minimize the failure risk of the CW lines, APS places each unit in a pro-active maintenance outage where the extent of distress in each unit is considered in determining spacing of the outages (e.g., 2 to 3-yr spacing for the highly distressed Unit 3 CW line). For each outage, a certain amount of resources are allocated for condition assessment and repair, the use of such resources are optimized by detailed planning of all activities to take place during the outage, as follows:

- Pre-Outage: APS first determines the duration of the outage and the duration that can be allowed for condition assessment and repair of the CW lines. The recommendations made by SGH after the previous outage are reviewed, and a preliminary scope of work is determined based on the available outage duration. Tentative repair drawings (e.g., CFRP liner) and specifications are prepared by SGH for bidding based on "potential" repair pipes based on previous inspection results. APS receives the bids from the contractors and secures resources required to perform the estimated scope of work well in advance of the outage, including contingencies. Several meetings are held close to the outage date with involved parties (e.g., EM inspectors, SGH, APS, repair contractors) to finalize the scope of work and schedule.

- Outage: CW lines are dewatered immediately and prepared for safe access for EM (if recommended) and visual and sounding inspections within the first three days. EM inspection results become available within two days following inspection for evaluation by SGH in combination with the visual and sounding inspection results, including comparison with the results from previous years to determine progression of distress. During this evaluation, the repair contractor is on standby on site. SGH provides a list of pipes in the order of descending repair priority, APS finalizes the number of pipes to be repaired, and SGH issues repair drawings for construction. SGH provides field engineering support throughout the repairs for verification of compliance with repair drawings and specifications. The pipeline is returned to service after SGH verifies by laboratory testing that the repairs have cured sufficiently.
- Post-Outage: SGH prepares a detailed report of all construction and testing activities, including results of tension tests performed on CFRP witness panels made during construction to verify that the installed materials meet the properties considered in design. The report also includes recommendations for the following outage. A debriefing meeting is held soon after the outage to review the successes in the completed outage and discuss what can be improved for the next outage.

CONCLUSIONS

This paper presents the condition assessment, failure risk analysis, and repair methods used to minimize risk of failure for highly distressed CW lines made of PCCP, and shares the experiences gained from this program over multiple outages. The following are concluded:

- Failure risk of pipelines even with widespread severe distress can be minimized by performing pro-active maintenance programs that include thorough planning, proper selection and use of condition assessment, failure risk analysis, and repair methods, and evaluation of results based on sound engineering judgment.
- Condition assessment, failure risk analysis, and repair prioritization of highly distressed pipelines requires a combined evaluation and correlation of data obtained from multiple inspection methods such as EM inspections, verification of EM results by external inspection, internal visual and sounding, and failure risk analysis. Consideration of only one source of data may not minimize the risk of failure, considering for example that EM inspection results could not be verified by external inspection in some instances in Cholla.
- Severely distressed pipes with nearly all wires corroded away were identified by analysis of the EM signals by the inspection company. Failure risk and repair priorities can be assigned to these pies using the results of internal inspection results of the pipeline accompanied by limited external inspection of a sample of highly distressed pipe.

- Repair of PCCP with CFRP liners is an effective method even for severely distressed pipelines as they can be designed as a standalone system without any contribution from the host pipe, and installed quickly in many non-continuous pipe segments within a short period of time without requiring external access. Successful CFRP repairs require design according to the current state of the art presented in AWWA Draft Standard for CFRP Renewal and Strengthening of PCCP, including proper termination details and other special details, and also continuous engineering field support throughout CFRP installation for verification of compliance with repair drawings and specifications.

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Condition Assessment of Sanitary Sewer Lines Using Acoustic Inspection

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Abstract

Sanitary sewer overflows (SSOs) continue to be an issue for collection system managers, and performing additional cleaning without knowledge of the pipe condition results in a significant waste of resources. Rapid acoustic inspection technology can quickly determine the extent of blockage in a pipe and enables the user to deploy cleaning resources much more effectively. Municipal wastewater utilities struggle to effectively manage the vast underground network of pipes that handle the transportation of raw sewage through our nation's cities and towns. Capital investment needs for wastewater and stormwater collection systems are estimated at $298 billion over the next 20 years (ASCE, 2013), further squeezing operating budgets. Condition based maintenance (CBM) can assist by efficiently targeting maintenance to locations prior to an overflow or other failures. This requires cost effective/timely information to implement. A CBM program for collection system operations can substantially lower operating costs, but until recently obtaining the required assessment information was cost prohibitive. There are two key factors influencing a CBM program's viability: inspection cost, and the fraction of pipes requiring maintenance. A novel acoustic inspection technology (recently commercialized and evaluated by the EPA) is an enabling technology that allows for economical pre-cleaning assessment of sanitary sewer lines. The acoustic inspection technology is cost effective with inspections costing 1/10th the cost of CCTV inspections. In addition, based on acoustic inspection (and validated by CCTV during pilot projects) on average 50-70% of the pipes in a system do not need immediate maintenance or further detailed inspection. It's important to note that the acoustic inspection does not replace CCTV, it helps to prioritize where and when to use more expensive CCTV resources. Using acoustic inspection to prioritize cleaning operations could provide a breakthrough enhancement for moving collection systems maintenance towards efficient CBM programs. Economic analysis and productivity measurements will be presented that evaluate the effectiveness of using a preliminary inspection tool. Multiple case studies will be discussed, while also evaluating the substantial financial and operational impacts of using acoustic inspections to prioritize cleaning operations.

INTRODUCTION

Wastewater utilities face the daunting challenge of maintaining their collection systems in compliance with state and federal regulations. To be effective, utilities must be constantly vigilant in maintaining their collection systems in order to minimize sewer line blockages that push wastewater out of manholes and onto streets, public/private property and waterways (Figure 1).

Figure 1. Sanitary sewer overflow frequently caused by undetected blockages

Recognizing the potential public health hazard, the US Congress tasked the EPA to report on issues associated with combined sewer overflows (CSO) and sanitary sewer overflows (SSO) (EPA, 2004). CSOs occur in combined sewer systems (wastewater and storm water) and SSOs occur in systems with only wastewater.

The EPA estimates that each year more than 9,000 CSOs occur releasing 850 billion gallons of sewage and more than 25,000 SSOs occur releasing an additional 10 billion gallons of sewage. The EPA estimated cost to municipalities is staggering: "$50.6 billion required to reduce CSO by 85% by volume" and "$88.8 billion required to control SSOs over the next 20 years." Based in part on these findings, the EPA has become more aggressive in enforcing its zero tolerance for overflows issuing both Administrative Orders and judicial Consent Decrees. The risk to municipal utilities for non-compliance is significant ranging from fines and court mandated agreements to restricting growth and rescinding operator licenses.

Historically, using condition based inspection to determine where and when to deploy collection system cleaning resources has not been economically feasible. Visual manhole inspections rarely determine that a segment needs to be cleaned. Grease buildup and root infiltration accumulate from the top of the pipe downward. So by the time grease or roots cause a change in the wastewater flow – the pipe blockage is significant and the time to react is immediate – hours not days. Existing pipe inspection methods are either too cost prohibitive for widespread use or provide inadequate condition assessment. The Sewer Line – Rapid Assessment Tool (SL-RAT®) is an onsite inspection device developed by InfoSense with the support of Charlotte Mecklenburg Utilities (CMU). This technology was developed by Dr. Ivan Howitt while a faculty member at UNC Charlotte. This university research was motivated and sponsored by CMU. CMU is a valuable strategic partner contributing both extensive engineering knowledge and access to their collection system.

The diagnostic capability of this device allows cleaning requirements for pipe segments to be economically prioritized prior to conducting cleaning operations. Using this methodology requires significantly less resources than is required for current maintenance practices. This provides the opportunity to rethink using condition based maintenance as a viable tool for deploying cleaning resources, and can both improve maintenance quality and reduce unnecessary maintenance operations.

The wastewater industry needs a paradigm shift in how they approach collection system maintenance. Specifically, a move from maintenance based on excessive cleaning to a program based on directed cleaning using smart inspection, i.e., condition based maintenance (CBM) program. Operators understand the need, but are limited based on both current inspection tools and the lack of alternative maintenance programs which they can buy into.

TECHNOLOGY DESCRIPTION

The SL-RAT® exploits the similarities and difference between water and sound transmission through a sewer line segment in order to diagnose the pipe's blockage. This novel methodology is based on measuring the signal received from an active acoustic transmission through a segment, Figure 2. The sound wave generated at the transmitter propagates in the air gap above the flow from the speaker to the receiving microphone located at the adjacent manhole. Segment lengths exceeding 250m (800 ft) have been successfully evaluated. An important practical aspect of this methodology is that both the speaker and the microphone are placed just within the opening of the manhole and never come in contact with the wastewater flow. The acoustic transmitter generates sound waves just below the entrance to the manhole which naturally couple into connecting sewer line segments, whether the depth of the manhole is 1m (3 feet) or greater than 10m (30 feet). The acoustic receiver measures the acoustic plane wave from the transmitted signal in order to evaluate the condition of an entire sewer line segment.

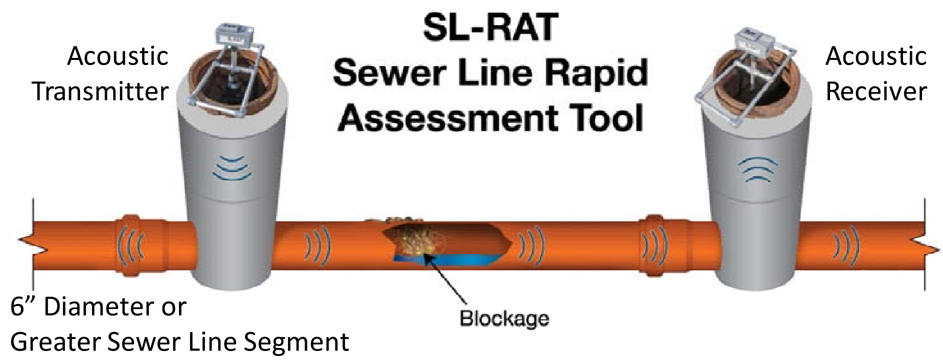


Figure 2. Concept and operation of the SL-RAT® Acoustic Inspection System.

A clean segment is a natural acoustic waveguide. As illustrated in Figure 2, commonly encountered sanitary sewer defects, such as roots, grease and sags naturally absorb or reflect acoustic energy. These defects change a segment's acoustic properties and produce a measurable impact on the received signal at the microphone, i.e., the segment's acoustic fingerprint (SAF). Each segment has an individual SAF representative of its current state. The SAF changes over time as the condition of the sewer line segment varies. The equipment uses the SAF to make a blockage assessment, i.e., an estimate of the aggregate blockage within the segment between the acoustic transmitter and acoustic receiver.

Using the blockage assessment, a segment can be classified onsite as requiring cleaning or not. Characterizing the acoustic equipment's ability to classify was an objective of a recent joint pilot project conducted by CMU and InfoSense (Howitt, 2010). Both CCTV videos and SL-RAT SAFs were obtained both prior to and after cleaning. Comparison between the CCTV blockage assessment to the SL-RAT blockage assessment was then possible. Figure 3 shows that the points in the scatter plot are correlated and lie in the lower right triangle implying the acoustic blockage assessment is a conservative estimator of the CCTV blockage assessment. In the figure, a CCTV threshold of 3 (0-Obstructed and 10-Clean) is used to classify sewer line segments. Using this CCTV threshold, 86% of the sewer line segments are classified as not requiring cleaning. Correspondingly, the SL-RAT® correctly classifies 61% of the segments as not requiring cleaning and no segments requiring cleaning are misclassified. By design, using acoustic inspection is a conservative estimator of the blockage condition (no data points in the upper left-hand quadrant of the chart).

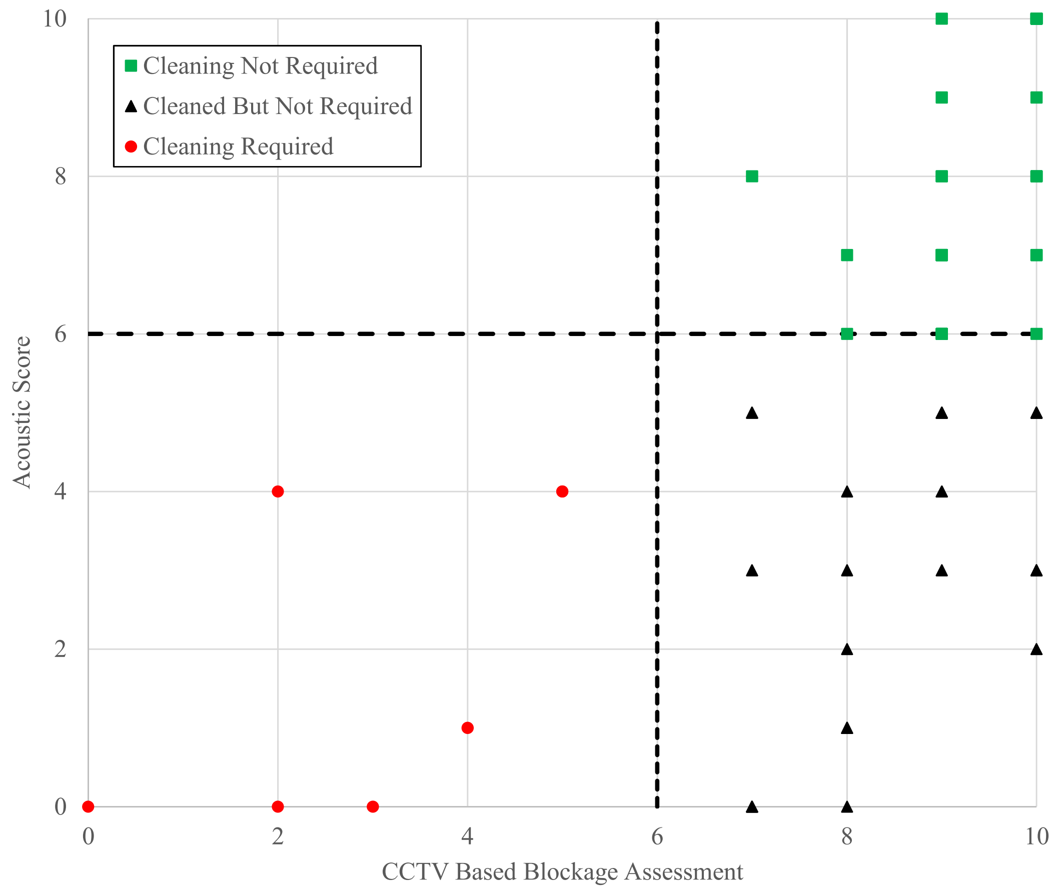


Figure 3. Scatter plot comparing blockage assessments for acoustic inspection versus CCTV

There are several limitations to acoustic inspection technology worth noting. The physical shape of the pipe can impact acoustic scores, particularly pipe sags. Figure 4 shows impact of pipe sags on the air gap. When there are partial sags, the acoustic score becomes much more sensitive to flow. In the extreme case of a full pipe sag (no continuous air gap between manholes), the acoustic measurement will always show a "blocked" pipe (acoustic score of 0). While it is beneficial to locate these kinds of defects, once a pipe sag is located, that particular segment will always provide an acoustic score of 0 until the defect is repaired. Using acoustic inspection in these types of pipes will still provide a conservative approach to determining the need to clean, but systems with a large number of pipe sags will lean toward lower acoustic scores.

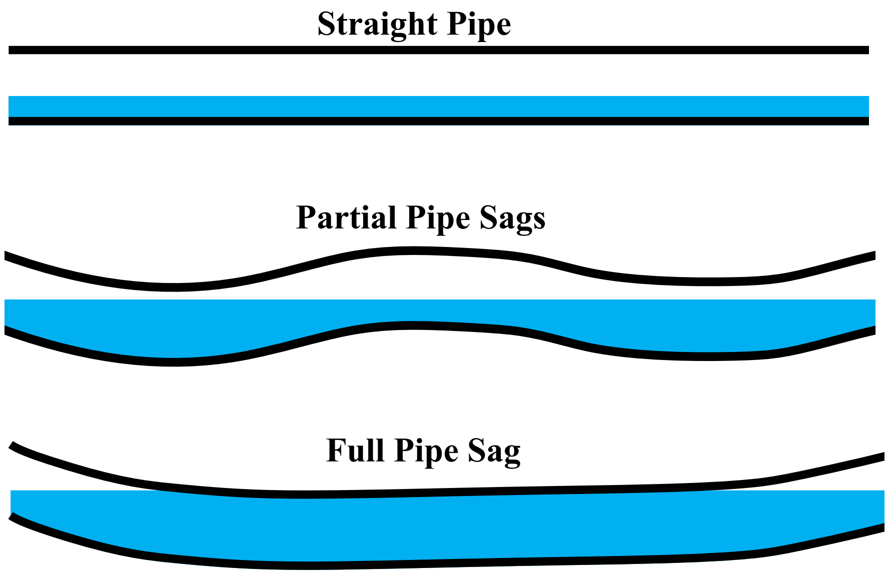
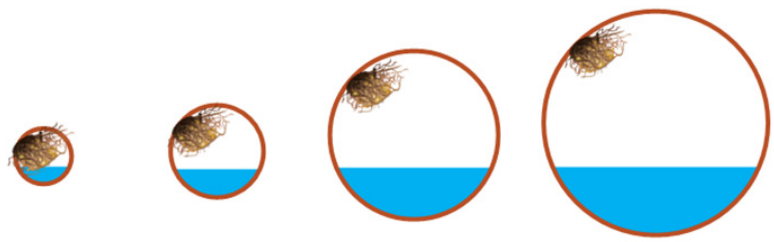


Figure 4. Impact of Pipe Sags on Air Gap Between Manholes

Figure 5 shows the impact of pipe diameter on available surface area for a given size blockage. The acoustic inspection device is intended for use with 150 to 300mm pipe diameters (6" to 12"). As the pipe diameter increases, there is more surface area for sound to travel around the blockage, and so larger diameter pipes will tend to give higher acoustic scores. While it is possible to use acoustic inspection devices on larger diameter pipes, the preferred operating range is 150-300mm in diameter (6"-12").



Diameter	6 inches	10 inches	18 inches	24 inches
Total surface area (sq.in)	28.3	78.5	254.5	452.4
% blocked	89%	48%	32%	29%

Assumes pipe is ¼ full with flow, obstruction is 18 sq. inches

Figure 5. Impact of Pipe Diameter on Open Surface Area in a Pipe

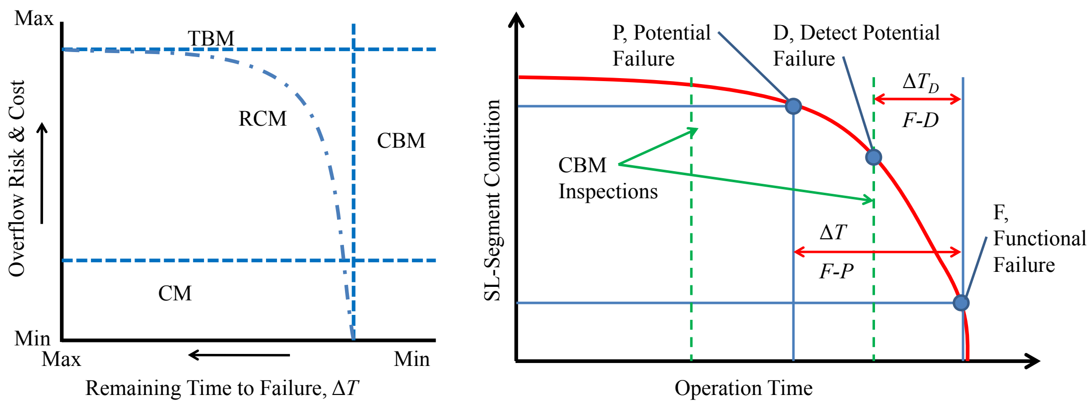
CONDITION BASED MAINTENANCE PROGRAM

Maintenance policies for wastewater collection systems' cleaning operations are currently a combination of fixed interval maintenance, i.e., Time-Based Maintenance (TBM) and reactive maintenance, i.e., Corrective Maintenance (CM). Figure 6(a) illustrates the optimal region of application for each strategy. The horizontal axis represents the remaining time to failure with values decreasing towards the right. The vertical axis represents the relative risk and the cost associated with a pipe segment overflow. To illustrate, vandalism can lead to overflows, e.g., dumping leaves in a manhole. Since vandalism is an unlikely event and the time to failure is short, a CM program is the only option. A TBM program is appropriate in areas where periodic cleaning interval is required and can be reliably estimated, e.g., areas with high grease restaurants. In these areas there is a high risk and the time interval to failure can be predicted.

A preponderance of grease and root blockages occur over a sufficiently long time interval, suggesting a CBM program is optimal. From Wiseman et. al.,

> "Condition based monitoring is defined as: an identifiable physical condition which indicates that a functional failure is either about to occur or in the process of occurring. In this process, the items are inspected and left in service on the condition that they meet specified performance standards. The frequency of these inspections is determined by the potential failure (P-F) interval, which is the interval between the emergence of the potential failure and its decay in to a functional failure."

Developing an overall maintenance policy that balances the maintenance strategies is the goal of Reliability Centered Maintenance (RCM) (Moubray 1997). RCM allocates cleaning resources based on optimizing the cost and risk associated with overflows.



(a) (b)

Figure 6. (a) Regions of optimal application for four maintenance strategies (Lehtonen 2006); (b) Relationship between inspection and P-F in a CBM based maintenance strategy (Moubray 1997 and Wiseman et. al.).

Historically, using condition based inspection to determine where and when to deploy collection system cleaning resources has not been economically feasible. The available inspection technologies are either cost prohibitive or provide inadequate information. Preliminary acoustic inspection provides a clear condition assessment directly correlated with the cleaning requirements. Acoustic inspection requires significantly fewer resources compared to normal maintenance, i.e., acoustic inspection has been shown to be substantially cheaper than cleaning or CCTV inspection at \$0.15/ft (EPA, 2014). This provides the opportunity to rethink using condition based maintenance as a viable tool for deploying cleaning resources. This approach can improve maintenance quality, reduce unnecessary maintenance operations and, at the same time, reduce costs.

The previous discussion motivates the value proposition for implementing the Sewer Line Condition Based Maintenance (SL-CBM) program based on acoustic inspection. Figure 6(b) illustrates the concept and the challenges with implementation. The graph in the figure is a standard P-F curve (Moubray 1997 and Wiseman et. al.) depicting a graceful degradation in a pipe segment with the condition assessment graph representative of a grease or root mode of failure. Point P represents the initial time performance degradation can be detected and Point D represents the time performance degradation is detected based on the acoustic CBM inspection schedule. Point F represents the operation time at which the sewer line pipe segment functionally fails, e.g., the blockage is sufficient to cause an overflow. Each pipe segment has a unique P-F curve governed by underlying factors influencing its failure rate. The goal of the SL-CBM is to estimate the CBM inspection and maintenance times to ensure maintenance is scheduled and conducted prior to the pipe segments failure at a significant cost savings over current maintenance programs.

Acoustic inspection is an essential tool in developing an effective SL-CBM cleaning program. The graph in Figure 7 provides a hypothetical comparison of the cost effectiveness between three cleaning programs. The purpose of the comparison is to illustrate the flexibility and trade-offs available in designing a cleaning program based on acoustic inspection. From previous analysis (Howitt, 2010), an operations performance goal of 2 overflows/100 miles/year requires 76% of the collection system to be maintained annually. The corresponding one standard deviation below the mean requires 45% of the system to be maintained.

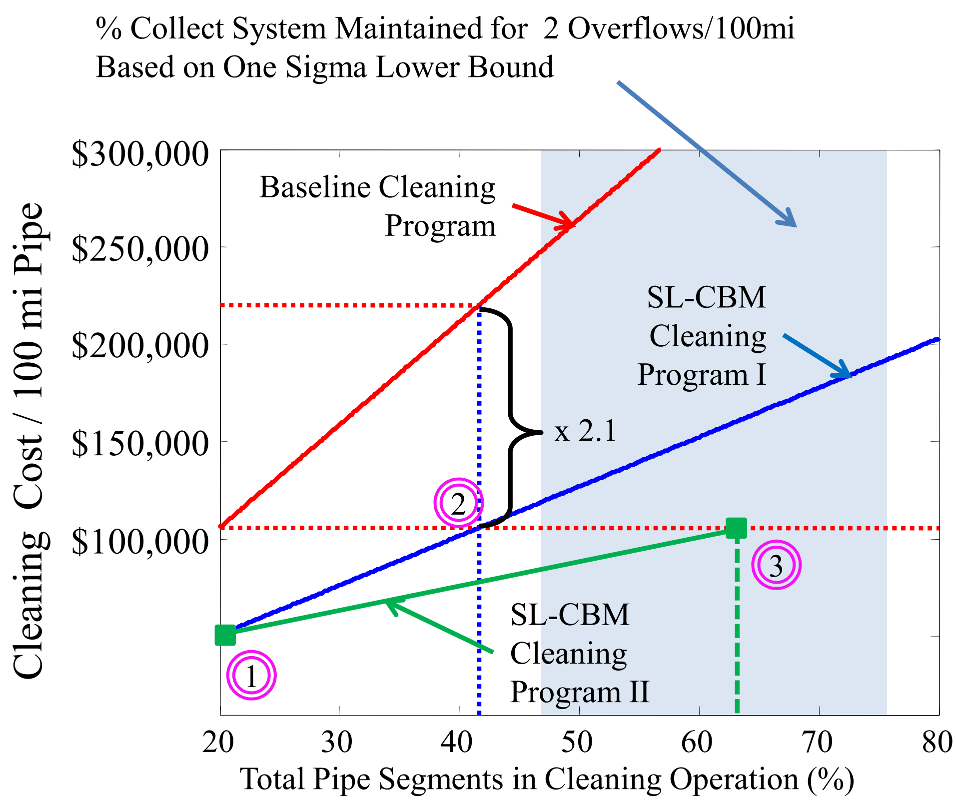


Figure 7. Collection System cleaning policy comparison between Baseline Cleaning Program with two different Condition Based Maintenance programs (SL-CBM Program I & SL-CBM Program II) which use acoustic inspection to prioritize cleaning operations.

In Figure 7 at Point-1, for the baseline cleaning policy, 20% of the pipes are cleaned annually at $1/ft. For this cost analysis we only consider the cost of cleaning and do not include any cost for post-cleaning inspection with CCTV. The other two policies are based on using the acoustic inspection tool prior to cleaning. For these two policies, 20% of the pipes have been inspected and, based on their acoustic blockage assessment, only 7.8% are estimated to require cleaning resulting in over a 50% cost savings. Only segments which are diagnosed as essentially clean are removed from the cleaning operations resulting in no impact on collection system performance.

Next we look at keeping the budget fixed at the 20% annual baseline cleaning cost and look at two different cleaning and inspection policies using acoustic inspection. For SL-CBM I, we continue using the policy that only essentially clean line segments are removed from the cleaning operations based on their acoustic blockage assessment. This allows us to acoustically inspect 41.6% of the collection system and based on the acoustic blockage assessment only 16.2% require cleaning, Point-2. SL-CBM II takes a different approach by switching modes to focus on only cleaning the pipe segments which are in immediate need, i.e., only clean if diagnosed with a significantly low acoustic score. The policy transfers more resources towards inspection rather than cleaning, allowing 63.4% of the collection system to be acoustically inspected, with an estimated 14.1% cleaned and with an estimated 9.5% diagnosed as having a significant blockage assessment by the acoustic inspection tool, Point-3. This suggests that by using acoustic inspection, over 60% of the collection system can be maintained annually at a comparable cost as a 20% annual baseline cleaning program. This achieves the goal of maintaining the collection system between 45% and 76% without increasing the annual cost. Acoustic condition assessment is used to cost effectively target cleaning resources to locations with a higher likelihood to cause overflows.

The previous discussion provides a general assessment of SL-CBM program using acoustic inspection. Variations in implementation are examined under the assumption that overflows are equally likely within the collection system. We next turn to evaluating the SL-CBM cost versus performance impact taking into account the historical spatial overflow patterns within an actual collection system.

The approach is to evaluate the cost associated with the SL-CBM program based on establishing a new collection system cleaning program to achieve a desired number of overflows/ 100mi of linear pipe, i.e., the performance goal

$$P_T = \frac{O_T}{N_T} \tag{1}$$

where O_T is the total number of overflows within the collection system based on the utilities maintenance program and N_T is the number of 100mi lengths of pipe within the collection system. The evaluation model is derived to evaluate the total cost of the maintenance program

$$C_T = \sum_i C_i \quad (2)$$

where C_i is the cleaning operation cost for the i^{th} Region. The value of C_i is evaluated for two cases: Cleaning Only program with no acoustic inspection and SL-CBM program based on acoustic inspection, i.e.,

$$C_i = C_C\, A_i N_i \quad \text{[Cleaning Only no acoustic inspection]} \quad (3)$$

$$C_i = [C_C\, A_i + C_I I_i] N_i \quad \text{[SL-CBM with acoustic inspection]} \quad (4)$$

where C_C is the cost to clean 100mi length of pipe and C_I is the cost to acoustic inspect 100mi length of pipe with the SL-RAT. A_i is the fraction of the i^{th} Region cleaned and I_i is the fraction of the i^{th} Region inspected acoustically. For the SL-CBM program, A_i is determined based on the acoustic threshold used to discriminate between pipe segments requiring cleaning and those that do not. The relationship between the acoustic threshold and the fraction of pipe segment cleaned, D, is derived based on historical acoustic inspections and the relative occurrence of blockage assessments. The acoustic thresholds evaluated are the same as those used in the previous studies and the values D for the acoustic thresholds are given in Table 2. Then using $A_i = D\, I_i$, the SL-CBM cost for the i^{th} Region's is

$$C_i = [C_C\, D + C_I] I_i\, N_i \quad \text{[SL-CBM with acoustic inspection]} \quad (5)$$

Next, the total number of overflows is given by

$$O_T = \sum_i O_i\, N_i \quad (6)$$

where O_i is the number of overflows/100mi of linear pipe in the i^{th} Region and N_i is the number of 100mi lengths of pipe in the i^{th} Region.

The overflows/100mi for the i^{th} Region can be estimated prior to the effect of the acoustic inspection. These values are estimated based on the linear regression and are given in Table 1 and are used in evaluating O_i.

Table 1. Typical Overflow Temporal and Spatial Data Summary

	Entire Collection System	Region 0	Region I	Region II	Region III
Total number of overflows	4386	0	922	1157	2307
Number of square miles	526	136	233	77	80
Number of linear miles of pipe line	4261	437	1869	874	1081
Overflow/100mi rate of change	-0.6	0	0.4	-1.4	-1.7
Overflow/100mi	7.3	0	5.1	8.2	13.4

To evaluate O_i, the new maintenance program performance needs to be evaluated in terms of the former maintenance program. Using this approach, the i^{th} Region's overflow/100mi is modeled by

$$O_i = R_C A_i - R_C F_i + R_i \quad \text{[Cleaning Only no acoustic inspection]} \tag{7}$$

$$O_i = R_C I_i - R_C F_i + R_i \quad \text{[SL-CBM with acoustic inspection]} \tag{8}$$

where for the SL-CBM program it is implicit that the fraction of the i^{th} Region cleaned is $A_i = D I_i$. R_C is the rate of change in the number of overflows/100mi based on the change in the fraction of the area maintained (cleaned or inspected). F_i is the fraction of the i^{th} Region cleaned under the former maintenance program and R_i is the overflow/100mi in the i^{th} Region based on the former maintenance program.

Table 2. Parameter Values Used in Evaluating the SL-CBM Program Cost-Performance

Parameter	Value
Fraction of pipe segments cleaned, *D*, for acoustic threshold 1	0.17
Fraction of pipes segments cleaned, *D*, for acoustic threshold 3	0.23
Fraction of pipes segments cleaned, *D*, for acoustic threshold 5	0.34
Ratio acoustic inspect cost to cleaning cost, C_I/C_C	0.09
Rate of change in the number of overflows/100mi to the change in the fraction of the collection system area maintained, R_C	-7.8

R_C is an important parameter in evaluating the effectiveness of a maintenance program. It specifies the rate in achieving the performance goal based on either cleaning more pipe segments or by improving the selection process for targeting cleaning resources to the pipe segments requiring cleaning. R_C is estimated from previous data (Howitt, 2010). For the results presented in this paper, the value is considered a constant. This provides a first order approximation. R_C is likely to be region dependent and dependent on the maintenance program followed, i.e., for the SL-CBM program, R_C will be impacted by the acoustic threshold selected. As the acoustic threshold is increased, the number of pipe segments scheduled for cleaning increases. In addition, the cleaning targets pipe segments which are increasingly cleaner. Therefore, the value of R_C will initially rapidly improve with a diminishing improvement as the acoustic threshold increases. This relationship has not been established and therefore is not used in evaluating the results in the paper.

The desired model for relating the total cost in terms of the overflows/100mi of linear pipe is obtained by combining equations (1) through (8)

$$C_T = \frac{N_T K}{R_C} P_T + K \sum_i F_i N_i - \frac{K}{R_C} \sum_i R_i N_i \tag{9}$$

where $$K = C_c \quad \text{[Cleaning Only no acoustic inspection]} \tag{10}$$

$$K = C_c D + C_I \quad \text{[SL-CBM with acoustic inspection]} \tag{11}$$

Looking at the three terms in equation (9) provides insight into the model. The first term, $(N_T\, K/R_C)P_T$, provides the head room savings based on the performance goal being greater than zero overflows/ 100mi. The second term, $K\sum_i F_i\, N_i$, is the cost of meeting the former maintenance performance using the new maintenance program and the third term, $(K/R_C)\sum_i R_i\, N_i$, is the cost of mitigating the reported overflows based on the new maintenance program.

By relating equations (10) and (11), the mechanism for achieving substantial cost savings using the SL-CBM over the Clean-Only program can be readily evaluated. For the same performance goal, the SL-CBM will be less expensive than the Clean-Only program given

$$C_c > C_c\, D + C_I$$
$$\frac{C_I}{C_c} < 1 - D \tag{12}$$

As discussed in previous sections, the cost of acoustic inspection is, conservatively, less than a tenth the cost of cleaning. In addition, the fraction of pipe segments not requiring servicing ($1 - D$) are at least 50% and often significantly greater. The inequality in equation (12) is well met, leading to substantial cost savings for the same performance goal.

The total maintenance program cost, C_T, is evaluated using equations (9) through (11) based on varying the performance goal, P_T. Graphs of the evaluation are depicted in Figure 8. The cost in Figure 8 has been normalized by the estimated cost for cleaning 20% of a collection system ($0.2N_T C_C$). This normalization removes the uncertainty associated with cleaning cost, C_C. The parameter values used in evaluating the equations are summarized in Tables 2 and 3. Three SL-CBM programs are compared based on using different acoustic thresholds. These results are compared to the program based on Cleaning-Only.

From Figure 8, a SL-CBM program using an acoustic threshold of 3, results in a performance of four overflows/ 100mi without increasing cost. To achieve the same performance with the Clean-Only program requires over 3 times the cost.

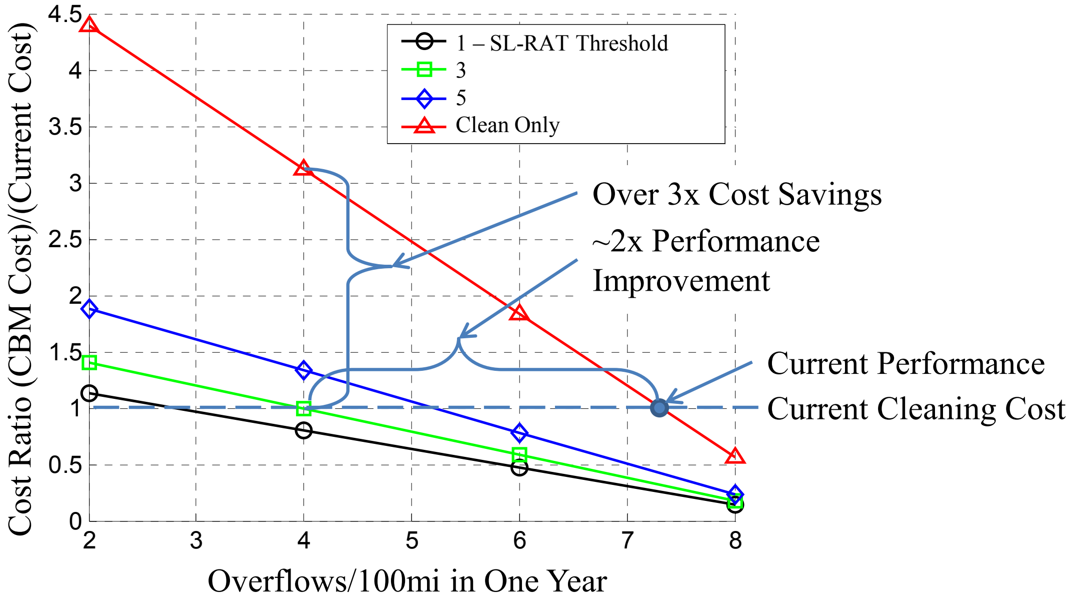


Figure 8. Cost versus performance evaluation for the SL-CBM. Cost is evaluated based on the ratio between the cost of the SL-CBM program with the cost of the current cleaning program, i.e., Cleaning 20% of the Collection System/Year. Performance is based on the number of overflows per 100 miles in a year.

CONCLUSION

In summary, the basic operational principles of a new pipe inspection technology have been explained and shown through several examples from Charlotte-Mecklenburg Utilities, (CMU), multiple ways that active acoustic inspection can significantly improve the maintenance cost and performance of gravity-fed wastewater collections systems.

Additionally, multiple operational benefits of acoustic inspection relative to existing alternatives were covered. Acoustic inspection does not require confined space entry and does not contact the wastewater flow making it safer to operate. It also does not require the support of cleaning equipment, provides the blockage assessment in 3 minutes or less, has been practically operated in a typical wastewater collection field environment under a variety of conditions, and can be easily operated by a field crew of two operators.

The results of multiple pilots and field studies conducted by several cities show the efficacy, the economics, and the operational advantages of the SL-RAT device. The efficacy of acoustic inspection technology was highlighted in a study conducted to correlate CCTV video with the aggregate blockage measurement provided by acoustic inspection. Acoustic inspections were shown to successfully detect blockages within a pipe segment and to provide acceptable resolution for delineating when pipe cleaning activity should take place and when it should not. These same field studies estimated the cost of operating active acoustic inspection equipment and found that the acoustic inspection is very economical compared with the cost of cleaning and/or CCTV operations.

Finally, we looked at extending the use of acoustic inspection technology to enable the establishment of a Condition Based Maintenance (CBM) program for gravity-fed collection system pipe maintenance. An example was illustrated using data to extrapolate that for a system targeting 2 overflows/100 miles/year, implementing an SL-RAT-enabled CBM program could reduce cleaning costs by 50% or more. The improvement compounds as more resources are shifted to the relatively cheaper task of acoustic inspection and away from the relatively expensive and partially wasteful task of scheduled pipe cleaning. The financial benefit comes through better focusing cleaning crews on blocked pipes and away from cleaning pipes that do not need cleaning. These results were extended further to develop a performance model which illustrates mathematically that using acoustic inspection data as part of a CBM program can provide significant benefits to wastewater system operators by producing BOTH a significant positive impact on overflow performance as well as system maintenance costs.

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Development of Performance Index for Stormwater Pipeline Infrastructure

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Abstract

Stormwater infrastructure asset management is becoming increasingly popular in the United States, with emergent government regulations and knowledge of the risk posed by deteriorating stormwater pipelines to the environment. Performance assessment of a stormwater pipe is an essential aspect of utility asset management plans. This paper presents a weighted factor framework to determine the performance of stormwater pipes. A list of about 50 parameters effecting the performance of stormwater pipelines was prepared based on literature review, study of existing stormwater asset management plans and feedback from utilities. The list was divided into essential and preferential parameters, due to a lack of readily available pipe parameters with utilities. A two-level hierarchical representation of degradation of stormwater pipeline infrastructure was developed, constituting five failure modules and the essential parameters. On the basis of survey replies obtained from ten utilities across the EPA regions, the essential parameters were combined into a performance index. The index is a scale of one to five, similar to National Association of Sewer Service Companies' (NASSCO) Pipeline Assessment Certification Program (PACP) grading system. Furthermore, a prototype of the performance index was developed using real utility data.

1.0 Introduction

Stormwater infrastructure asset management is a relatively new concept (Grigg 2012). Stormwater infrastructure is usually considered a subsidiary of wastewater infrastructure, which is evident from the fact that American Society of Civil Engineers categorize it under wastewater in the 2013 Infrastructure Report Card (ASCE 2013). The decision tools developed for wastewater pipelines are generally adopted directly for stormwater pipeline maintenance (Betz 2013). However, this is inadequate, considering the differences between stormwater and wastewater pipelines. Wastewater pipe systems are constructed with high quality, have longer pipe sections, and are installed at lower depths. The wastewater pipes have a constant flow pattern. They flow both under pressure and gravity. The stormwater pipes construction quality relies upon the contractor since visual inspection of the asset is not a mandate after construction in all jurisdictions. The stormwater pipes are often built of short sections and are installed at shallower depths when compared to wastewater pipes. Stormwater contains varied debris than can result in surface wear of the pipes. The flow pattern in a stormwater pipe is directly linked to the precipitation in the area and hence, is not constant. The stormwater pipes usually remain empty for a portion of the year. The major cause for deterioration of the wastewater pipes is the internal attack by acids associated with sewage; whereas, stormwater pipelines are relatively clean, and are predominantly damaged by external factors (Micevski et al. 2002). Given these differences, an effort was made in this research to develop a performance index specific to stormwater pipeline infrastructure.

2.0 Background

A separate stormwater pipeline system is an independent network, which conveys the water that flows over land or impervious surfaces, as a result of snowmelt or rainfall, to nearby streams (USEPA 2015). A stormwater pipeline refers to the length of pipe between manholes or a node in this research. A node can be a junction, stormwater inlet or outlet.

The performance of a stormwater pipeline is its ability to convey the stormwater discharges in accordance with hydraulic design requirements, in a manner that it causes minimum damage to the environment while maintaining sound structural integrity (Mitchell Shire Council 2012). A poorly maintained stormwater system can lead to flooding of the neighborhood, damage of public infrastructure and loss of human life (Jacobs et al. 1993).

Stormwater Pipe Materials

Stormwater pipes can be classified based on material type into rigid (concrete, vitrified clay), semi-rigid (corrugated metal, brick) and flexible (high-density polyethylene (HDPE), polyvinyl chloride (PVC)) (ASCE Standard 2006). The use of clay and brick pipes has been discouraged by utilities for new installations (Betz 2013). Based on the study of literature and interaction with stormwater utility personnel, the following understanding of pipe materials is presented (Bishop and Sertich 2013; ASCE Standard 2006; Sinha et al. 2008; National Corrugated Steel Pipe Association (U.S.) 2008).

Metal pipe: The structural strength of a metal pipe depends on the accurate placement and compaction of the backfill. Improper installation of joints can compromise the condition of the backfill due to exfiltration of water or infiltration of backfill into pipe. Lack of a minimum cover over the top of the metal pipes combined with high external loading can lead to failure of metal pipes. Corrosion, abrasion, repair of interior coating and clogging are common reasons for maintenance of metal pipes. The rotting of the invert and invert lifting are common failures observed in corrugated metal pipes with age. The optimal function of all stormwater pipes, including metal pipes requires accurate calculation of the required design storage volume.

Plastic pipe: Plastic pipes are quite flexible and their long term performance also depends on the selection and compaction of backfill. It is crucial to control deflection of plastic pipes during installation, to avoid open connections and holes, which can result in settling or washouts. Thermal expansion and debris build up are other reasons for failure of plastic stormwater pipes.

Concrete pipe: Improper designs of pre-cast pipes, which do not consider site specific conditions or adverse weather, can result in poor structural condition. Improper site preparation can compound the failure of stormwater pipe by leading to differential settlements. Debris is a common reason for periodic cleaning of concrete pipes.

Stormwater Pipe Failure Modes

Failure modes are defined as a type of failure which occur within a pipe (Sinha and St. Clair 2014). The failure modes found in stormwater pipelines are listed in Table 1 below (Integrated Science and Engineering Inc 2013; MnDoT Bridge Hydraulics 2013; Burns & McDonnell Engineering Company Inc 2010).

Table 1. Stormwater Pipeline Failure Modes

Failure Mode	
Cracks and Fractures	Longitudinal, circumferential, multiple, helical/Spiral
Deformed, collapsed and broken pipes	Punctures, breaks, deflection/compression, dent, collapse
Displaced and open joints	Gaps in joint, horizontal and vertical joint offset, dropped invert/misalignment
Surface damage	Exposed reinforcement and aggregate
Defective connections	Mid line or break in type connections without manholes
Debris, silt and obstructions	Foreign obstructions that reduce the hydraulic capacity of the pipe
Infiltration	Water infiltration within pipe structure
Exfiltration	Water exfiltration to surroundings
Settlement	Scouring/erosion of bedding material
Encrustation, scale and physical damage	Erosion of pipe lining, chipping, puncturing, scaling, spalling, rusting, collapse, corrosion
Water Level	Level of water in the pipe on any normal day above design level

From the above list, it can be concluded that most of the stormwater failure categories are addressed in NASSCO's PACP (NASSCO 2003). However, there is scope to alter the detailed defect codes and defect scores to better suit stormwater pipelines. For example, the hole is a major defect for stormwater pipes; its codes can be expanded to include the intrusions from other utilities, sink holes, etc. The severity of the defect varies significantly between stormwater pipes and wastewater pipes. For example, the visible reinforcement in a concrete pipe needs to be downgraded as stormwater is not as highly corrosive as wastewater.

3.0 Performance Data Structure Development

Many factors directly and indirectly affect the performance of a stormwater pipe network (Singh et al. 2007). Table 2 shows the list of factors considered in literature to determine the condition of stormwater pipelines.

Table 2. Factors Affecting Stormwater Pipe

Reference	Factors
(Micevski et al. 2002)	Diameter, Material, Soil type, Exposure classification
(Singh et al. 2007)	Age, Diameter, Material, Length, Traffic load, Land use, Maintenance frequency
(Tran et al. 2007); (Tran et al. 2009)	Age, Size, Soil type, Location, Pipe depth, Pipe slope, Tree count around the pipe, Tormwaite Moisture Index, Structural condition, Hydraulic condition
(Harvey and McBean 2014)	Age, Diameter, Material, Length, Soil type, Pipe depth, Pipe slope, Pipe thickness

A comprehensive list of about 100 parameters that affect performance of wastewater pipelines was already identified in literature (Angkasuwansiri and Sinha 2013). Due to similarities of storm sewers to gravity sanitary sewers, this list was taken as a basis and tailored to stormwater pipelines based on the developed understanding of stormwater pipe design and installation, stormwater pipe materials, failure modes and feedback from utilities. Table 3 and Table 4 below list the 50 parameters, which were divided into four classes based on their characteristics, namely, Physical/Structural, Environmental, Operational/Functional and Others (Angkasuwansiri and Sinha 2013). The parameters were further broken down into essential and preferential data (Angkasuwansiri and Sinha 2013).

Table 3. Essential Stormwater Pipe Performance Parameters

Classification	Parameters
	Essential
Physical or Structural	Age, Diameter, Length, Material, Shape, Depth of cover, Slope, Joint type, Associated structures, Lateral connections count, Bedding condition, Trench backfill condition, Design life, Design storm, Function
Environmental	Soil type, Groundwater table, Location, Loading condition (Dead Load), Loading condition(Live Load), Average precipitation intensity, Average precipitation duration
Operational or Functional	Average flow velocity, Minimum flow velocity, Overflow frequency, Surcharging, Inflow and infiltration, Exfiltration, Debris level , Sedimentation level, Smell or vermin level
Other	Post installation condition, Maintenance method, Inspection/CCTV record

Table 4. Preferential Stormwater Pipe Performance Parameters

Classification	Parameters
	Preferential
Physical or Structural	Thickness, Pipe coating type, Pipe lining type
Environmental	Frost penetration, Soil corrosivity, Proximity to trees, Proximity to utilities, Surrounding temperatures, Record of extreme event
Operational or Functional	Design velocity, Presence of stagnant water within structure
Other	Capital cost, Annual operational and maintenance cost, Renewal record, Failure record, Complaint record

4.0 Performance Index Development

Method

The parameters in Table 3 were combined mathematically to determine the performance of stormwater pipe. Due to availability of limited field data and since this is an introductory research, weighted factor method was chosen to determine the performance of stormwater pipes. The weighted factor model is a form of multi attribute or multi criteria analysis. It involves evaluation of all attributes that are relevant to the problem, allocation of a score to each attribute based on a rating scheme and allocation of weight to each attribute based on its relative importance. The weighted factor method is shown in Equation 1 below:

$$Y = \sum_{i=1}^{n} w_i \times p_i$$

Equation 1. Weighted Factor Model

Where, Y = Performance Index; p_i = Input parameter scores; w_i = Weights

Layout

The stormwater performance was analyzed in five modules, namely, capacity module, blockage module, overload module, surface wear module, and structural module. The capacity module indicates that the design capacity of the stormwater pipe is not sufficient to hold the current stormwater runoff in the location resulting in overflows and surcharging. Blockage module indicates that the pipe has sufficient capacity but the extraneous material entering the stormwater pipe is resulting in flow disruption. Surface Wear Module indicates that the surface of the pipe is deteriorating due to spalling, wear, mineral deposits, corrosion etc. Overload Module indicates that the pipe structure or shape is deteriorating. Structural Module includes all other structural defects not defined by surface wear and load modules like crack, fracture, broken, hole, joint, lining, etc. Each module is then separated into individual parameters that affect the module (Table 5).

Table 5. Layout of Stormwater Pipe Performance Index

Module	Parameters
Capacity	Overflow frequency, Surcharging, Inflow and infiltration, Exfiltration, Average precipitation intensity, Average precipitation duration, Average flow velocity, Soil type, Location
Blockage	Debris level, Sedimentation level, Lateral connections, Inlet or outlet is attached or if pipe changes direction or if cross bore is present, Smell or vermin level, Length, Diameter, Slope, Minimum flow velocity
Overload	Condition as per NASSCO's PACP (related to pipe shape), Loading condition (Dead Load), Loading condition (Live Load), Depth of cover, Bedding condition, Trench backfill, Ground water table, Exfiltration, Shape
Surface Wear	Condition as per NASSCO's PACP (related to pipe surface wear), Maintenance method, Age, Material, Shape, Average flow velocity
Structural	Condition as per NASSCO's PACP (relate to pipe structural condition), Age, Material, Diameter, Joint type

Weights - Data Collection

The expert opinion on the weights was collected through survey. The survey was sent to over 50 stormwater utilities across the 10 EPA regions. The list of 50 utilities was prepared based on available contacts from a previous research project (Betz 2013) and the list of stormwater utilities published by Campbell (2013). A minimum of three utilities were contacted from each of the 10 EPA regions to fill the surveys. A pairwise comparison matrix was created for the five modules to determine the relative weights. Also, the utilities were asked to provide the significance value (Very High, High, Medium, Low, Very Low) for each of the performance parameters affecting a particular module. The overall weight of a parameter is derived by multiplying its normalized weight with the weight of the corresponding module. About 10 replies were obtained from the survey, eight of which were survey inputs, while the other two were insights that helped improve the layout. No replies were received from EPA region 1 and EPA region 2, which have minimum to no stormwater utilities (Campbell 2013).

The weights indicate that the performance of a stormwater pipeline is highly affected by it structural condition, followed by blockage and capacity (Table 6). The weights for surface wear, overload and structural modules add up to 60 percent and the weights of capacity and blockage module add to 40 percent. This supports the idea that hydraulic condition of stormwater pipeline is crucial for its performance.

Table 6. Weights of modules calculated from the survey

Module	Weight (%)
Capacity	18
Blockage	22
Surface Wear	8
Overload	10
Structural	42

Input Parameter Scores

Each of the parameters were rated on a scale of one to five, where one implies excellent condition and five implies very poor condition. The rating scheme was developed on the basis of the literature (NASSCO 2003; Sinha and Angkasuwansiri 2010) and inputs from utility experts.

Performance Scale

Presently, many stormwater utilities use NASCCO's scale, while a few others use "Good-Poor-Failed" rating, a 0-5 scale, and 1-10 scale to gauge the condition of their assets (Betz 2013). The stormwater performance scale in this research was developed on the basis of NASSCO's PACP scale of one to five condition rating. Table 7 below indicates the calculated pipe score and the corresponding performance (St Clair 2013).

Table 7. Performance scale corresponding to pipe score (*Y*) generated in Equation1.

Pipe Score (*Y*)	Performance Scale	Description
1.0-1.5	1	Excellent
1.5-2.5	2	Good
2.5-3.5	3	Fair
3.5-4.5	4	Poor
4.5-5.0	5	Immediate Attention Required

Percentage Reliability

The data that is fed into the performance index determines the reliability of the output of the index. If the number of input parameter records available is less than the number of parameters actually supported by the model, the accuracy of the model output is reduced. Also, the accuracy of the input parameter records determines the accuracy of the model output (St Clair 2013). The source of the parameter was used to determine the confidence in the parameter in this research (Table 8) (Angkasuwansiri and Sinha 2013).

Table 8. Parameter Confidence Scale

Parameter Source	Confidence Scale (CS)
Direct Record	5
Derived Indirectly	4
Educated Guess (High Confidence)	3
Educated Guess (Medium Confidence)	2
Educated Guess (Low Confidence)	1

The equation below was used to determine the percentage reliability of the performance index.

$$Percentage\ Reliability = \frac{Obtained\ Parameters\ (= n)}{Requested\ Parameters\ (= 38)} * \frac{\sum_{i=1}^{n} CS_i}{(5 * n)} * 100$$

$$Percentage\ Reliability\ = \frac{\sum_{i=1}^{n} CS_i}{1.9}\ \%$$

5.0 Performance Index Application

To illustrate the application of the stormwater pipeline performance index, a case study utilizing real utility data from Utility A is presented. This stormwater utility serves a population of approximately 500,000. There are approximately 500 miles of pipe, 300 miles of open drainage, 21,000 inlets, outlets, manholes and junction boxes, 550 culverts and 100 detention ponds in the utility. Up to seven essential stormwater parameters were obtained for each pipeline from the utilities geodatabase. Additionally, the soil data was derived from United States Department of Agriculture web soil surveys. (USDA NRCS 2013). The loading condition (Live Load) was determined by measuring the proximity of the pipe to the major roads and highways. The GIS files of the county's major roads and highways was obtained from their Department of Transportation website (Colorado Department of Transportation). The equation below indicates the performance index generated using the nine parameters.

*Performance Index = 0.0169 * Soil Type + 0.0277 * (Inlet/Outlet/Pipe changes direction/Cross bore present) + 0.0238 * Length + 0.0796 * Diameter + 0.0266 * Slope + 0.0116* Loading Condition (Live Load) + 0.089 5*Age + 0.021 4* Shape + 0.1080 * Material*

The parameter scores (Table 9) were tailored with the help of the utility personnel to better represent their assets.

Table 9. Input performance parameter scores

Parameter	Range	Score
Soil Type	high plasticity clay (Group D)	5
	low plasticity clay (Group C)	4
	fine sand and silt (Group B)	3
	Coarse sand (Group A)	1
	Gravel (Group A)	1
Inlet/Outlet/Pipe changes direction/Cross bore present	Yes	5
	No	1
Length	Greater than 500 ft.	5
	400 ft. - 500 ft.	4
	300 ft. - 400 ft.	3
	200 ft. - 300 ft.	2
	Less than 200 ft.	1
Diameter	6 ft. - 12 ft.	5
	12 ft. - 18 ft.	4
	18 ft. - 24 ft.	3
	24 ft. - 36 ft.	2
	Greater than 36 ft.	1
Slope	Less than 0.5% or Greater than 5%	5
	0.5% - 1%	4
	1% - 2%	3
	2% - 5%	1
Loading Condition (Live Load)	Heavy - 20 ft. from major road/railway	5
	Medium - 50 ft. from road/railway	3
	Light - Greater than 50 ft. from road/railway	1
Age	Significantly greater than design life	5
	Very Highly greater than design life	4
	Moderately greater than design life	3
	Equal to design life	2
	Less than design life	1
Shape	Round	5
	Elliptical	4
	Arch, Rectangular	2
	Trapezoidal, V shaped	1
Material	Reinforced Concrete	5
	HDPE	4
	Polymeric Coated Corrugated Metal	3
	Aluminized Corrugated Metal	2
	Galvanized Corrugated Metal	1

Results

The performance index for each pipeline in the utility was evaluated using Equation 1. Table 10 below indicates the performance distribution of the 18,352 pipes. The percentage reliability for the pipes varies between 5 percent and 24 percent.

Table 10. Overview of Stormwater Pipelines Performance of Utility A

Performance Scale	Description	No. of Pipe	Percentage of Pipe
1	Excellent	165	1 %
2	Good	1509	8 %
3	Fair	7800	43 %
4	Poor	8673	47 %
5	Immediate Attention Required	205	1 %

6.0 Conclusions

An effort was made in this study to assist stormwater utilities in answering one of the core questions of asset management, "What is the current state of my assets?" A data structure of about 50 parameters, with units, was developed to encourage systematic data collection and storage in utilities. A framework based on weighted summation method was developed to gauge the performance of stormwater pipelines. The utilities are encouraged to tailor the weights and parameter scores to represent their in-house assets before applying the performance index. A prototype performance index was developed for one utility using this framework. The parameters and the weights were retained, but the parameter ranges and scores were edited based on the inputs from the utility experts.

The significant challenge associated with this research was the presence of minimum literature specific to stormwater pipelines outside environmental affects, which was met by actively involving utility asset managers. The developed performance index is subjective, as it is based on only 10 survey replies and is not validated. In future, the study can be repeated with all the stormwater utilities in the country, incorporating validation techniques, preferably on-field or laboratory experiments. Future research can also aim at improving the understanding on failure mechanisms of stormwater pipe materials.

The framework developed in this research shall enable stormwater utilities to gauge the performance of their existing pipelines. The developed performance index shall aid stormwater utilities in explaining the current condition of their pipelines to the government officials and public, thereby increasing the monetary investments. The percentage reliability of the calculated performance index shall encourage stormwater utilities to maintain data records. In conclusion, this study is a stepping stone towards shifting the stormwater utility maintenance efforts from being reactive to proactive.

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Protocol for Water Pipeline Failure and Forensic Data Analysis

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Abstract

The key to implementing infrastructure asset management strategies is to have a comprehensive understanding of the asset performance, and how this performance changes over time. The rate of deterioration of pipes is affected by a number of factors. Although there are many condition assessment technologies and accelerated testing methods available, these techniques do not provide all of the required data on the factors effecting deterioration. Earlier efforts have been made to capture data on pipe failure and as described in the literature, these data have been used to develop of some prediction models. However, failure data alone are limited in terms of understanding pipeline deterioration. Forensic analyses of pipe samples extracted while in service as well as after failure are crucial in acquiring a comprehensive understanding of the rate of deterioration over time. Water sector utilities need feasible and sound protocols to analyze these pipe samples. This paper presents methods and protocols for failure and forensic analyses and collecting data on the deteriorated or failed pipe samples. These proposed protocols contain guidelines on capturing the environmental (soil, groundwater, climate, etc.), and structural (corrosion, fracture, fatigue, etc.) data for drinking water and wastewater pipelines with field and laboratory tests. Furthermore, a Web-based and GIS driven platform called PIPEiD (Pipeline Infrastructure Database) has been developed to collect and share the data produced following these failure and forensic analysis. The objectives of the PIPEiD is to unite the nation's water pipeline infrastructure data and information, to make it universally accessible and useful, and to provide access to the data sources, tools, and models that enable the analysis, simulation, visualization, and evaluation of the behavior of water pipeline infrastructure systems for advanced asset management. PIPEiD is envisioned to be "a Living Database Platform for Advanced Asset Management" addressing all three major asset management levels including strategic, tactical, and operational that will assist drinking water and wastewater utilities of all sizes to sustain targeted levels of service with acceptable risk.

INTRODUCTION AND BACKGROUND

In 2005, the USEPA Office of Wastewater Management held a collaborative working session with 140 water and asset management professionals from 12 countries. The group voted on their top 10 action item priorities. Among the action item priorities, #3 was "Development of a central depository of high quality data available to researchers" and #6 was "Develop uniform national standards for condition assessment and asset reporting." According to the 2008 Drinking Water Infrastructure Needs Survey, 60 % of the estimated needs were for transmission and distribution pipelines. The Clean Watershed Needs Survey indicated nearly 28% of estimated need was for sewer pipelines. There is an urgent need to establish a standard data model and centralized database for storing, updating, retrieving, exporting, importing, analyzing, and verifying water infrastructure performance data. The key for implementing asset management is a comprehensive, standardized, and centralized platform that will enable an enhanced understanding of the characteristics directly affecting pipeline lifecycle performance. There is a need to establish a standard data model and centralized database for storing, updating, retrieving, exporting, importing, analyzing, and verifying pipeline performance data for the asset management purposes of water and wastewater pipelines. Such advancements will help water utilities affordably develop and implement robust decision-support systems to better understand the condition of their assets and to predict life cycle management needs. PIPEiD is envisioned to provide a unified platform for the nation's water pipeline infrastructure data and information, to make it universally accessible and useful, and to provide access to the data sources, tools, and models that enable the analysis, simulation, visualization, and evaluation of the behavior of water pipeline infrastructure systems for more effective management of pipeline assets.

Understanding a pipe's life cycle and classifications of the varying failure modes and mechanisms helps in identifying the parameters that affect the pipes overall quality, condition, and/or performance. Collecting and analyzing these separate pipe parameters will provide a data structure. Previous research has determined and evaluated the list of parameters to be included in the data standard models for water and wastewater pipeline performance and failure (Halfway et al. 2006, Al-Barqawi and Zayed 2006, MacKellar 2006, Wood and Lence 2007, Sinha et al. 2008, Grigg 2009, Vemulapally 2010, Klainer at al. 2010, St. Clair and Sinha 2014). These studies suggest lists of parameters for data standard models and their relations will be examined for this task. With the guidance from the data standards committee, the initial list of parameters to be included in the data standard model which is pertinent to the water and wastewater will be defined and piloted with the PIPEiD Database and participating utilities. Current list of parameters and coding found in the literature and practice is insufficient to reflect the condition and rate of deterioration of the pipeline assets; additional extensive standardized data needs to be collected throughout the life cycle (including design, manufacturing, installation, commissioning, operation, maintenance, rehabilitation, and replacement) to achieve effective asset management. Without data standards to capture these crucial variables, data interoperability cannot be achieved. The research team will develop national data standards for asset inventorying and condition assessment of water and wastewater pipes, including Wood, Bronze, Silver, Gold, and Platinum data needs, augmenting the suggested data requirements within the WERF Report *Predicting the Remaining Economic Life of Wastewater Pipes* (SAM3R06) and covering physical, operational, environmental, and financial data. To achieve effective risk management, the research team will consider the business risk exposure (BRE) that is the likelihood of failure (LoF) multiplied by the consequence of failure (CoF), and a mitigation factor.

Key stages of data collection include: baseline, operational and maintenance (O&M), and forensic for which the methodologies and protocols for data collection will be developed. The data collection standards will comprehensively cover each life stage of the pipe, during which several categories of data can be captured: the physical data (material, diameter, age, location, etc.); O&M data (water pressure, quality, inspection reports, maintenance and renewal activities, etc.); environmental data (soil characteristics, groundwater table, frost penetration etc.); and financial data (capital cost, operation cost, depreciation, replacement value, etc.). Such comprehensive data collection will require the management and integration of vast amounts of data; the creation of a data standards model will greatly assist with automating such data management. The metadata standards will allow: the dynamic extraction of such data for aggregation into databases; interoperability across water and wastewater utility databases; and interoperability across other asset management support systems. Beyond the database design aspects pertaining to the storage of spatially referenced data describing the engineered water infrastructure, the data standards need to incorporate relationships between spatial, and even unstructured information sources of the infrastructure features themselves. An infrastructure for the upload of data, prior to ETL into the standard data model, must be designed with this in mind. On top of the infrastructure for data management, which consists of physical repositories for the data itself and standardized data structures, there must be overlaid a well-defined ETL rules and processes to automate assimilation of data into the model for analysis and decision making.

PIPE PERFROMANCE MODELS TO SUPPORT ASSET MANAGEMENT

Water pipeline performance prediction models provide decision support in asset management of these infrastructure systems. Various inputs regarding environmental, structural, functional, and economic factors are required to evaluate the performance of drinking water and wastewater pipeline infrastructure, as shown in Figure 1.

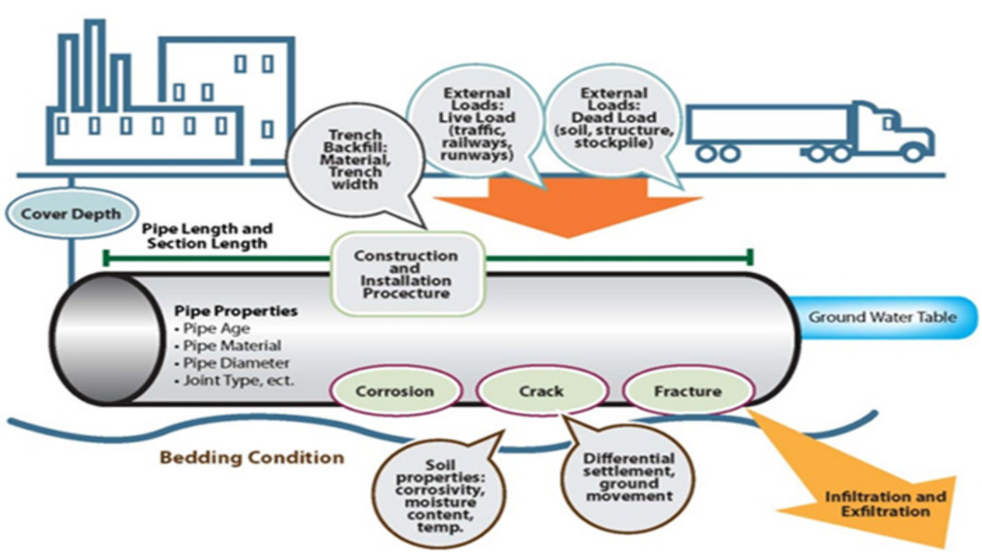


Figure 1. Various Parameters Affecting Performance of Water Pipeline Infrastructure

Water Pipeline Performance Prediction Models in Literature

Water performance models can roughly be divided in to four categories; statistical, probabilistic, advanced mathematics, heuristic. Statistical models are developed with historical data on pipe breaks to identify failure patterns, and they extrapolate these patterns to predict future pipe breaks (Malm et al. 2012, Wang et al. 2010, Wood and Lence 2009, Wang et al. 2009, Berardi et al. 2008, Kleiner and Rajani 2008). Probabilistic modeling depends on the use of statistical analysis to determine the probability or relative frequency of an event occurring (Rahman et al. 2014, Moglia et al. 2008, Davis and Marlow 2008, Davis et al. 2008, Dehghan et al. 2008). Advanced mathematical models are separated into two different classes: ANN (Jafar et al. 2010, Amaitik and Amaitik 2008, Geem et al. 2007, Al-Barqawi and Zayed 2006) and fuzzy logic (St. Clair and Sinha 2014, Fares and Zayed 2010). These two types of advanced mathematical models present approaches that have been frequently used for infrastructure deterioration. Heuristic models incorporate engineering knowledge rather than data parameters that affect a pipe to determine failure rates (Francis et al. 2014, Zhou et al. 2009). These modeling approaches can also be used in combination depending on the parameters and scope of these models (Azeez et al 2013, Kleiner et al. 2007, and Saridakis et al. 2006). More detailed reviews of the drinking water pipeline performance prediction models can be found in Rajani, and Kleiner (2001a, 2001b), St. Clair and Sinha (2012). More detailed reviews of wastewater pipeline performance models can be found in Tran (2007) and Ana and Bauwens (2010).

Water Pipeline Performance Prediction Models in Practice

St. Clair and Sinha (2012) conducted a comprehensive overview of current utility practices related to performance prediction models for drinking water pipes aiding in the understanding the gap between available models in literature and current utility practice in predicting water pipe performance. In order to determine current practices, nine utilities in the US, Australia and Canada, with significant activities of water pipe infrastructure management were contacted. The type of information requested within the survey included: types of inspection and condition assessment techniques used; prioritization of inspection, maintenance, repair, rehabilitation, and replacement (MRR&R); type of mathematical; methods used to generate condition curves; factors included within the condition curves; software used; associated costs in generating condition curves; and type of pipe condition and/or performance index. The exact performance prediction models utilized by utilities vary significantly. Generally, utilities use a type of long-term economic forecast model that is a tool designed to help the utility estimate the ‘‘economic life’’ of assets. These models provide decision support on planning for the maintenance and replacement of water pipe aiding to the total lifecycle cost analysis. Types of long-term economic forecast models presented through the survey consisted of Nessie Curves, Wave Rider Model, KANEW and the Computer Aided Rehabilitation of Water Networks (CARE-W), which has a Long-Term Planning (LTP) tool to estimate the long-term investment needs (St. Clair and Sinha 2012). NASSCO’s PACP code is typically used to record the defects of wastewater pipeline based on CCTV inspection. WERF Report SAM3R06a “Development of a Robust Wastewater Pipe Performance Index” presents a robust performance index for wastewater pipeline infrastructure system.

Limitations of Current Water Pipeline Prediction Models

A review of the current practice shows that many of the water utilities have been using some type of performance prediction model. However, the models developed lack robustness and reliability compared to the numerous models found in published literature (St. Clair and Sinha 2012). The models described in the literature were created with limited datasets, and there is a significant lack of methods and tools to evaluate and validate these models. Also, many of these models found in literature are relatively complicated for the average utility to apply to their own water pipeline infrastructure system and the accuracy of these models has been evaluated with very limited datasets (St. Clair and Sinha 2012). The lack application of methods to verify and validate the newly developed performance prediction models for water pipelines is one of the reasons creating this gap between academic literature and practice. Additional research is required to Verify and Validate (V_e & V_a) the deterioration prediction models to aid in bridging the gap between the models found in literature and the current utility practice. Following V_e & V_a framework offers techniques to increase applicability and accuracy of the water performance prediction models.

PIPE MODELS VERIFICATION AND VERIFICATION PROTOCOL

No matter the modeling paradigm or technique used to develop the model prediction, the model output will only predict the condition and/or performance of the real system as good as the models inputs and logic. Specifically, the accuracy of a model will be based on how close the models output corresponds to the real life scenario of the represented system. By its nature a model is an abstract of the system it represents. Abstraction and assumptions in creation of a model eliminates unnecessary detail and allows the developer to focus on the elements within the system that are important from a performance point of view. However, abstraction and assumptions may affect the accuracy of the model created. Model verification and validation are the primary processes for quantifying and building credibility in numerical models. Verification is the process of determining that a model implementation accurately represents the developer's description of the model and its solution. Validation is the process of determining the degree to which a model is an accurate representation of the real world from the perspective of the intended uses of the model (Sargent 2013). The proposed V_e & V_a framework relies on three domains of datasets - artificial, field, and experimental. The artificial dataset consists of the statistical data created in the acceptable ranges of the parameters used as the input for the model testing. The field dataset consists of water utility records and various data gathered from other sources regarding the environmental, structural, functional, and financial parameters. The experimental dataset consists of the results of the experiments run using pipe samples in the laboratory or field environment. These datasets are used to either run the model to observe the behavior or compare the outputs to assess accuracy of the model. Figure 2 represents the steps of the water pipeline model verification and validation framework. The V_e & V_a methodology have been utilized for WERF funded wastewater performance prediction model.

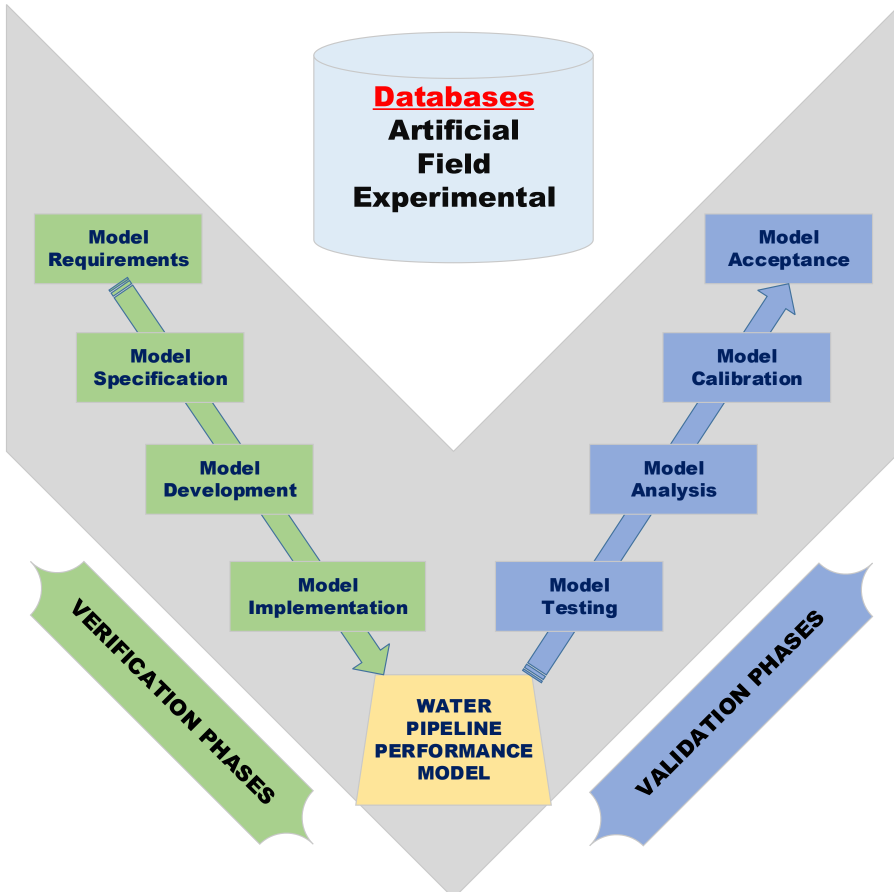
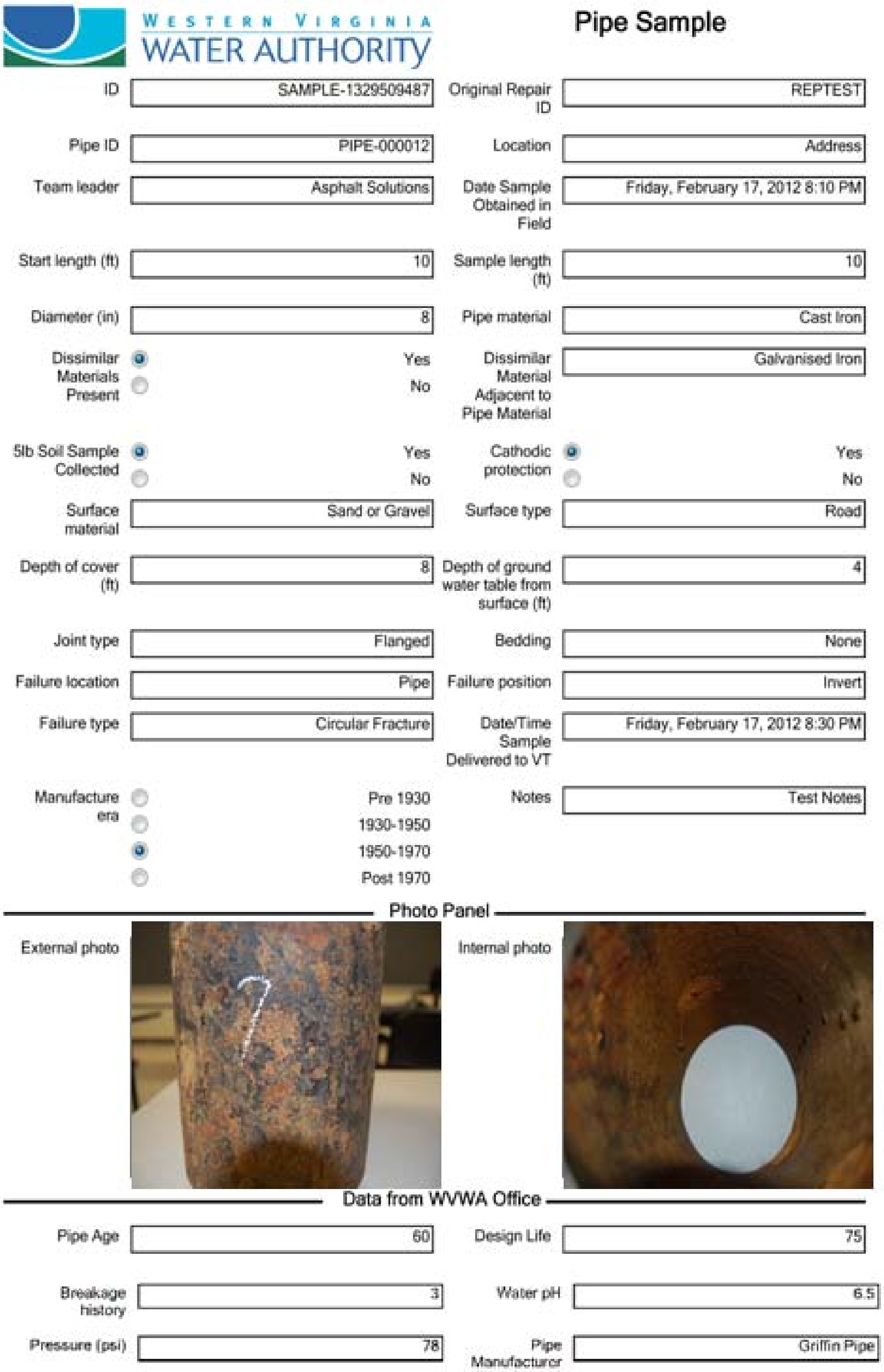


Figure 2. Model Verification and Validation Framework

WATER PIPELINE FAILURE AND FORENSIC DATA COLLECTION

The research team has been working closely with many water utilities including the Western Virginia Water Authority (WVWA), Roanoke, Virginia and Washington Suburban Sanitary Commission (WSSC), Laurel, Maryland on a project to collect aged pipe samples and capture field related pipeline data. The objectives of this research are to develop methods and protocols to better understand the effect of field variables on pipeline performance, and to fill the gaps in the data collected by the water utilities for pipeline asset management decision-making. In the pilot project, various field and laboratory test protocols were developed and implemented. The project aims to understand the various attributes of the pipelines such as: remaining wall thickness, tuberculation, corrosion, soil characteristics, soil corrosivity, loading, manufacturing defects, and joint failures. Additionally, these field and laboratory tests helped to understand what data should be collected. The improved understanding of the effects of these variables on pipeline performance will be used in validating and evaluating the pipeline performance prediction models. Figure 3 and 4 represent pipe sample sheets for field and laboratory data collection. The research team has collected data based on the proposed methodology from more than 100 pipe samples received from various water utilities across the United States.



WESTERN VIRGINIA WATER AUTHORITY

Pipe Sample

ID	SAMPLE-1329509487	Original Repair ID	REPTEST
Pipe ID	PIPE-000012	Location	Address
Team leader	Asphalt Solutions	Date Sample Obtained in Field	Friday, February 17, 2012 8:10 PM
Start length (ft)	10	Sample length (ft)	10
Diameter (in)	8	Pipe material	Cast Iron
Dissimilar Materials Present	Yes (selected) / No	Dissimilar Material Adjacent to Pipe Material	Galvanised Iron
5lb Soil Sample Collected	Yes (selected) / No	Cathodic protection	Yes (selected) / No
Surface material	Sand or Gravel	Surface type	Road
Depth of cover (ft)	8	Depth of ground water table from surface (ft)	4
Joint type	Flanged	Bedding	None
Failure location	Pipe	Failure position	Invert
Failure type	Circular Fracture	Date/Time Sample Delivered to VT	Friday, February 17, 2012 8:30 PM
Manufacture era	Pre 1930 / 1930-1950 / 1950-1970 (selected) / Post 1970	Notes	Test Notes

Photo Panel

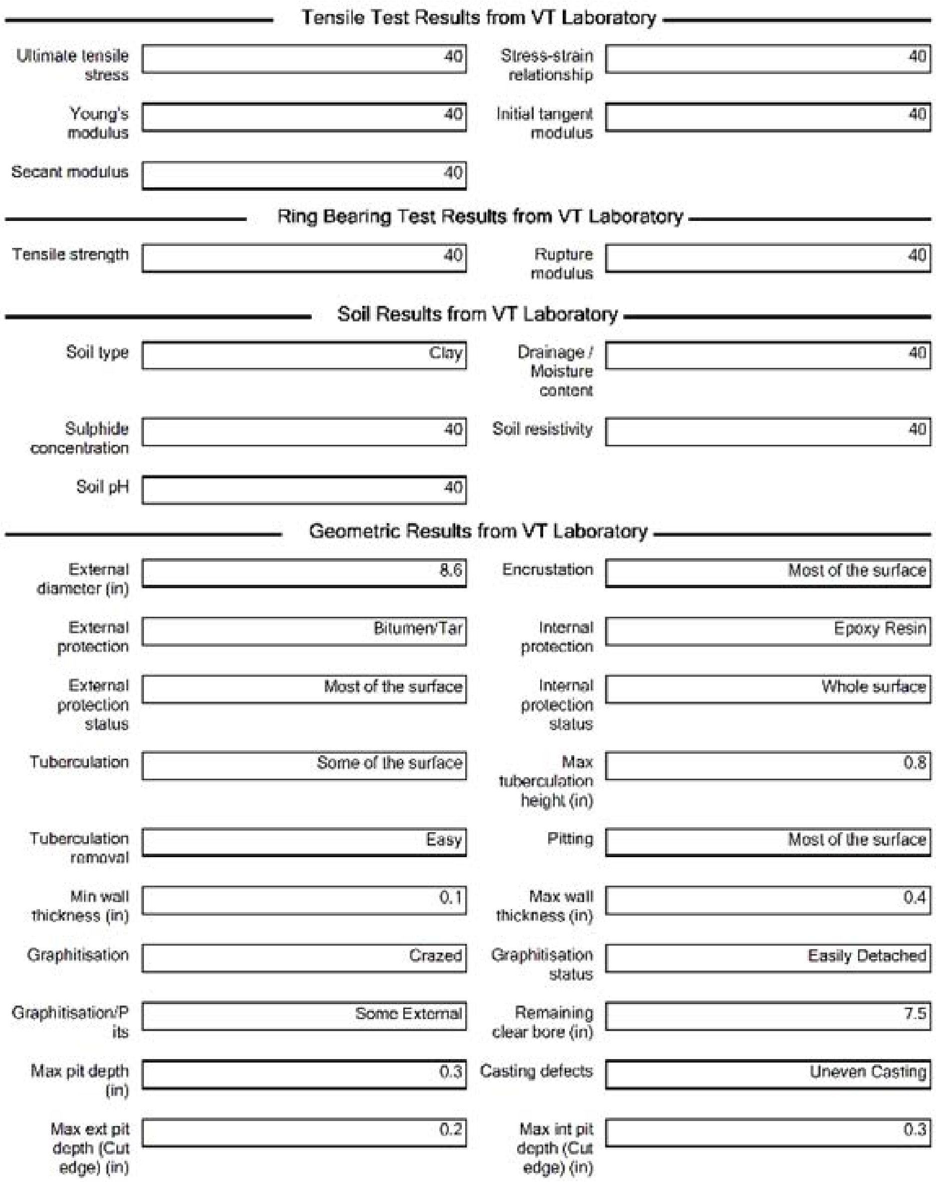
External photo

Internal photo

Data from WVWA Office

Pipe Age	60	Design Life	75
Breakage history	3	Water pH	6.5
Pressure (psi)	78	Pipe Manufacturer	Griffin Pipe

Figure 3. Water Utility Sample Data Collection Sheet – Field Information



Tensile Test Results from VT Laboratory

Ultimate tensile stress	40	Stress-strain relationship	40
Young's modulus	40	Initial tangent modulus	40
Secant modulus	40		

Ring Bearing Test Results from VT Laboratory

Tensile strength	40	Rupture modulus	40

Soil Results from VT Laboratory

Soil type	Clay	Drainage / Moisture content	40
Sulphide concentration	40	Soil resistivity	40
Soil pH	40		

Geometric Results from VT Laboratory

External diameter (in)	8.6	Encrustation	Most of the surface
External protection	Bitumen/Tar	Internal protection	Epoxy Resin
External protection status	Most of the surface	Internal protection status	Whole surface
Tuberculation	Some of the surface	Max tuberculation height (in)	0.8
Tuberculation removal	Easy	Pitting	Most of the surface
Min wall thickness (in)	0.1	Max wall thickness (in)	0.4
Graphitisation	Crazed	Graphitisation status	Easily Detached
Graphitisation/Pits	Some External	Remaining clear bore (in)	7.5
Max pit depth (in)	0.3	Casting defects	Uneven Casting
Max ext pit depth (Cut edge) (in)	0.2	Max int pit depth (Cut edge) (in)	0.3

Figure 4. Water Utility Sample Data Collection Sheet – Lab Information

Water pipe samples representing all the condition states are collection from WVWA and WSSC. A pipe sample collection protocol was established with WVWA and WSSC to collect the physical pipe samples and extensive data about these samples to aid in the model verification and validation process. Pipe samples would be collected during valve replacement and rehabilitation activities, or other activities that require a direct trench access to the pipelines. While these activities are conducted at the participating utilities, small sections of the pipes were cut and a 5 lb soil sample was also collected. Additionally, a standard data collection protocol was established to collect the required data pertaining to the collected samples. Table 1 represents the sample data collected through this protocol for pipe samples received from various water utilities and Table 2 shows evaluation of pipe sample based on expert opinion.

Table 1. Data Collected Protocol for Extracted Pipe Sample

Parameter	Unit	Pipe 1	Pipe 2	Pipe 3	Pipe 4	Pipe 5
Diameter	inch	6	6	6	6	1
Age	year	121	121	121	121	62
Design Life	year	120	120	120	120	50
Vintage	year	1891	1891	1891	1891	1950
Rehab (Lining)	yes/no	No	No	No	No	No
C Factor	c factor	75	75	75	75	50
Remaining Thickness	percent	100	100	100	100	50
Tuberculation	level	None	None	None	None	Moderate
Leak	yes/no	No	No	No	No	Yes
Pipe Break	yes/no	Yes	Yes	Yes	Yes	Yes
Break <5 Yrs Ago	yes/no	Yes	Yes	Yes	Yes	Yes
Defect Type	type	N/A	Severe	N/A	Severe	Extreme
R/R	type	N/A	None	N/A	None	None
Pressure Exceeded	occasion	Never	Never	Never	Never	Never
Pressure Surges	occasion	Never	Never	Never	Never	Never
Adequate Fire Flow	yes/no	Yes	Yes	Yes	Yes	Yes
Pressure Complaint	yes/no	No	No	No	No	No
Discolored Water	yes/no	No	No	No	No	No
Disturbances	yes/no	No	No	No	No	No
Flooding	occasion	Never	Never	Never	Never	Never
Live Load	road type	NonNHS	NonNHS	NonNHS	NonNHS	NonNHS
Material Type	type	CI	CI	CI	CI	GAL
Dissimilar Metals	yes/no	No	No	No	No	No
Cathodic Protection	yes/no	No	No	No	No	No
Stray Currents	yes/no	No	No	No	No	No
Soil Corrosivity	level	Moderate	Moderate	Moderate	Moderate	Low
Coating	yes/no	No	No	No	No	No

Table 2. Protocol for Evaluation of Pipe Sample based on Expert Opinion

Observed damage Level	Index range		Description	Action	Picture
Minor	Excellent	1	No noticable defects, some aging or wear may be visible	Do Nothing	
Minor	Good	2	Only minor deterioration or defects are evident	Monitor	
Moderate	Fair	3	Some deterioration or defects evident, function is not significantly effeced	Repair	
Major	Poor	4	Serious deterioration or defects in at least some portions of the pipe. Function is inadequate	Rehabilitate	
Failed	Failed	5	Extensive deterioration, barely functional or no function	Replace	

Expert Opinion (Heuristic Approach)

This decision framework is introduced to assist municipal engineers and planners to visually assess the condition of their water pipes (example, cast iron). It is important to note that the condition ratings assigned by the expert is solely subjective and these ratings are given resulting a visual inspection of the pipe. The expert opinion scale is set to be a condition index ranging from 1 to 5. A guidance document on the working principles of the model, and how to run the standalone model and the PIPEiD platform will be developed. Furthermore, outreach activities involving workshop, web-based education and training will be conducted to involve other water utilities to participate in this project and pilot the proposed model. These outreach activities will ensure higher acceptance and improved applicability for pipe performance models.

SUMMARY

The water sector pipeline performance models are strong tools in the utility asset manager's arsenal to conduct an efficient advanced asset management program. There are many models used in the condition evaluation and prediction, risk analysis, and renewal prioritization of drinking water and wastewater pipelines. However, many models have not been used by the water utilities because of the reliability of these models. This paper proposed a framework and field and laboratory data collection methodology that can be used to verify and validate models that are used for pipeline infrastructure asset management. By following the proposed framework to verify and validate the models that utilities are using, the accuracy and the confidence on the models can be greatly improved. Thus, the asset management programs at utilities would benefit greatly by utilizing the decision support models that are proven to be correct and accurate by using this framework. This framework is precise enough to prove the correctness and accuracy of many different types of models created for asset management decision support for water and wastewater pipelines. Also, the proposed PIPEiD centralized web-based platform will help to unite the nation's water sector pipeline infrastructure data and information for advanced asset management.

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Condition Assessment of Aging, Hard to Access Sewer Mains

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Abstract

Of the 1,300 miles of sanitary sewer lines owned and operated by the City of Baltimore, approximately 16 miles is 6-inch diameter, which is not included in the 2002 Consent Decree. While these pipes represent a small portion of the collection system, they generate a disproportionate number of service disruptions because they were constructed in the early 1900s using vitrified clay pipe and substandard construction methods by today's requirements. Faced with the question of how to deal with these problematic small sewer lines, Baltimore City decided to perform a comprehensive condition assessment, followed by the renewal of the assets depending on the results of the assessments. The 16 miles of 6-inch sewers was prioritized for condition assessment based on history of complaints and pending street rehabilitation projects. The intent of this project is to improve the level of service these assets can provide, thereby minimizing service disruptions and potential public health impacts associated with SSOs and basement back-ups.

BACKGROUND

The neighborhood of Roland Park is located within the Jones Falls Sewershed in northern Baltimore City and was developed between 1890 and 1920 by the Roland Park Company. The private developer, managed by planner Edward Bouton, laid out the street pattern, water lines, sewer and electric and sold property in a systematic and innovative way. The means and methods used by the Roland Park Company to construct the sewer lines differed from that used throughout the rest of Baltimore City and the standard details; the mains are predominantly 6-inch vitrified clay pipe laid in 3-foot segments. Instead of inserting manholes to change direction, the sewer lines were crimped to navigate bends. In addition, the manholes that were installed were 18-inches in diameter, instead of the standard 24 to 36-inch diameter required.

Eventually the water and sewer systems were annexed by the City of Baltimore, but the mains installed at the turn of the century remained in the existing condition and were never modified to conform to the rest of the system. Figure 1 shows the age distribution of the mains in the subject area and the average age is 83 years. As these assets have reached the end of their service life, open joints, cracked pipes, collapsed pipe have become all too common.

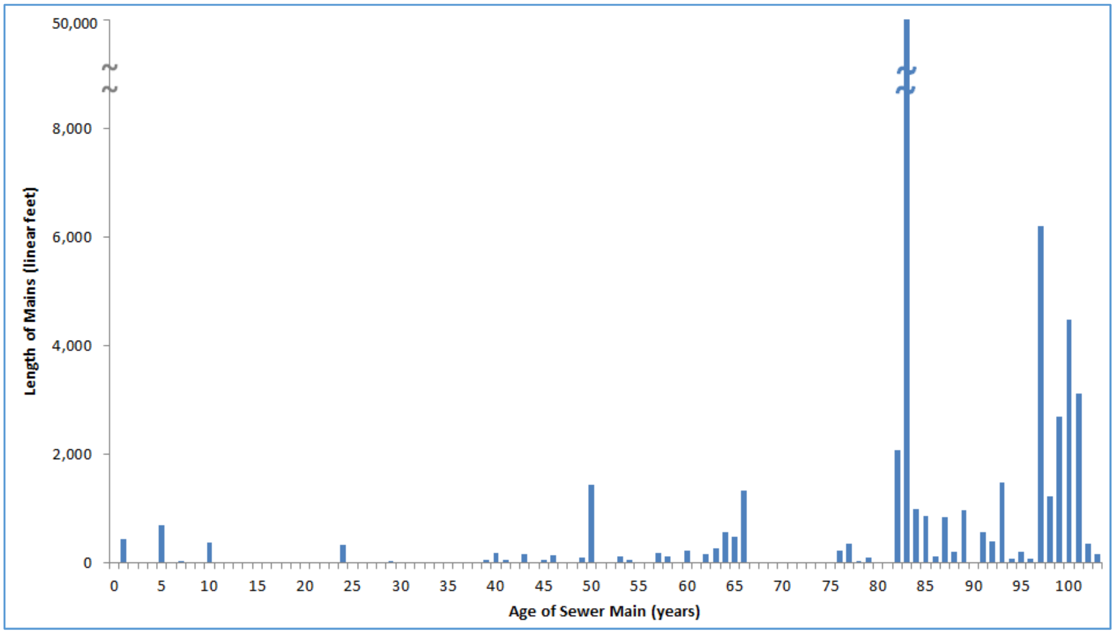

Figure 1: Age of 6-inch sewers in project area.

This neighborhood is also unique in that it was also one of the first "garden suburbs" in the country (Dickinson, 2014). The developers wanted a natural look and engaged a reputable landscape architect, Frederick Law Olmsted Jr., to be involved with every decision about what type of shrubs and trees to be planted. Wherever possible, mature trees were left in place and the neighborhood was developed around them. While this lead to the desirable garden suburb the Roland Park Company sought to achieve, it also lead to severe root intrusion in the aging, clay pipes that are still in service today.

Figure 2: Typical root-intrusion in 6-inch main

With approximately 16 miles, this area contains the largest concentration of 6-inch sewers in Baltimore City's collection system. This part of the system also accounts for a disproportionate percentage of complaints and service disruptions compared to the rest of the city. The number of sanitary sewer overflows (SSOs) and basement back-ups in the 6-inch sewers in the subject area has averaged 181 per 100 miles of sewer and 6 per customer per year, respectively, during the period of FY 2010 through FY 2014. On a yearly basis, the global median for SSOs is 2.7 per 100 miles of sewer (AWWA, 2014); the global benchmark for basement back-ups is 0.01 to 0.38 per 100 customers (Ofwat, 2010).

The Small Sewer Renewal Project was initiated by the Office of Asset Management (OAM) in 2014 to ultimately improve the level of service provided to the customers within the subject project area. Condition assessment began in 2014 and construction of renewal recommendations is anticipated to be complete in 2018. The following sections will discuss the means and methodology associated with implementing the Small Sewer Renewal Project.

COMMUNITY OUTREACH

While many residents within the project area have experienced multiple basement back-ups as a result of a failed sanitary system, and have been seeking a solution for years, they requested to be kept informed of all work that would impact the community and individual homeowners. Although the City has a utility easement, many of the sewers are located inside of the sidewalk, so it may appear that the sewer work is encroaching on private property. The Infrastructure Committee of the Roland Park Civic League was identified as the primary communication channel between Baltimore City Department of Public Works (DPW) and the residents. At the onset of the Project, DPW committed to keeping the public informed of pertinent aspects of the project including:

- the work schedule and limits
- potential traffic detours/disruptions
- pedestrian detours/disruptions
- sewer service disruptions

Prior to the start of the comprehensive condition assessment, a representative from the OAM attended the local Civic League meeting to introduce the plan to perform condition assessment and eventually repair or replace the 6-inch sewer mains in the neighborhood. The community was pleased to hear of the proposed work and receptive to the fact that there would be occasional disruptions as a result. Throughout the duration of the project, the DPW Community Liaison communicates regularly with the Civic League and responds directly to resident concerns. The Contractor is responsible for informing all affected parties of any disruptions in service, parking, or traffic due to the work. Typically planned disruptions are communicated in advance of the work by using signs along the roadway or door hangers.

PROJECT GOAL

The goal of the Small Sewer Renewal Project is to minimize the occurrence of mainline chokes, SSOs and basement back-ups associated with the 6-inch sewers in the project area, thus improving the level of service provided to these customers.

The Small Sewer Renewal Project will meet this goal in three phases:

1. Phase I - condition assessment of the full network of 6-inch sewers in the project area.

2. Phase II - design of repairs and rehabilitation based on the recommendations provided in the assessment.
3. Phase III - construction of the necessary improvements required to renew the assets and improve the level of service provided to the customers.

Phase I condition assessments of the 6-inch sewers is conducted in accordance with the National Association of Sewer Service Companies (NASSCO) Pipeline Assessment Certification Program (PACP). Performing the inspection in this manner ensures that the condition of the pipes are described in accordance with industry standard. In addition, defects can be easily identified by the engineer reviewing the closed circuit television (CCTV) documentation to assist in making final recommendations for repair or renewal; the PACP structural and operational scores are used by the engineer as a guide in assigning renewal recommendations.

Phase II consists of designing the repairs that resulted from the condition assessment. The detailed design is performed by licensed engineers and drawings are prepared and presented to the selected contractor. Phase III will be the construction of the designed repairs. If necessary, isolated repairs will be accelerated and performed by an urgent needs contractor in order to complete the condition assessment, mitigate a SSO or basement back-up, or restore service from a complete sewer failure. If not necessary, the project will be bid and performed systematically by the selected contractor.

As shown in Figure 3, the project area is divided into five project work areas to help prioritize the work, and communicate the scope and schedule with the local Civic League. The five work areas will remain as such through the assessment, design, and construction phases of the project. As the assessment is completed in Project Area 1, the design and construction will subsequently be performed. In the meantime, the assessment contractor will move into Project Area 2 and provide the results to the engineer to review.

The five project areas were prioritized based on the history of customer complaints; the area with the greatest number of complaints per footage of sewer mains is assessed first. Prioritizing the project areas in this fashion will hopefully address the most problematic sewers first and impact more customers during the early years of the project.

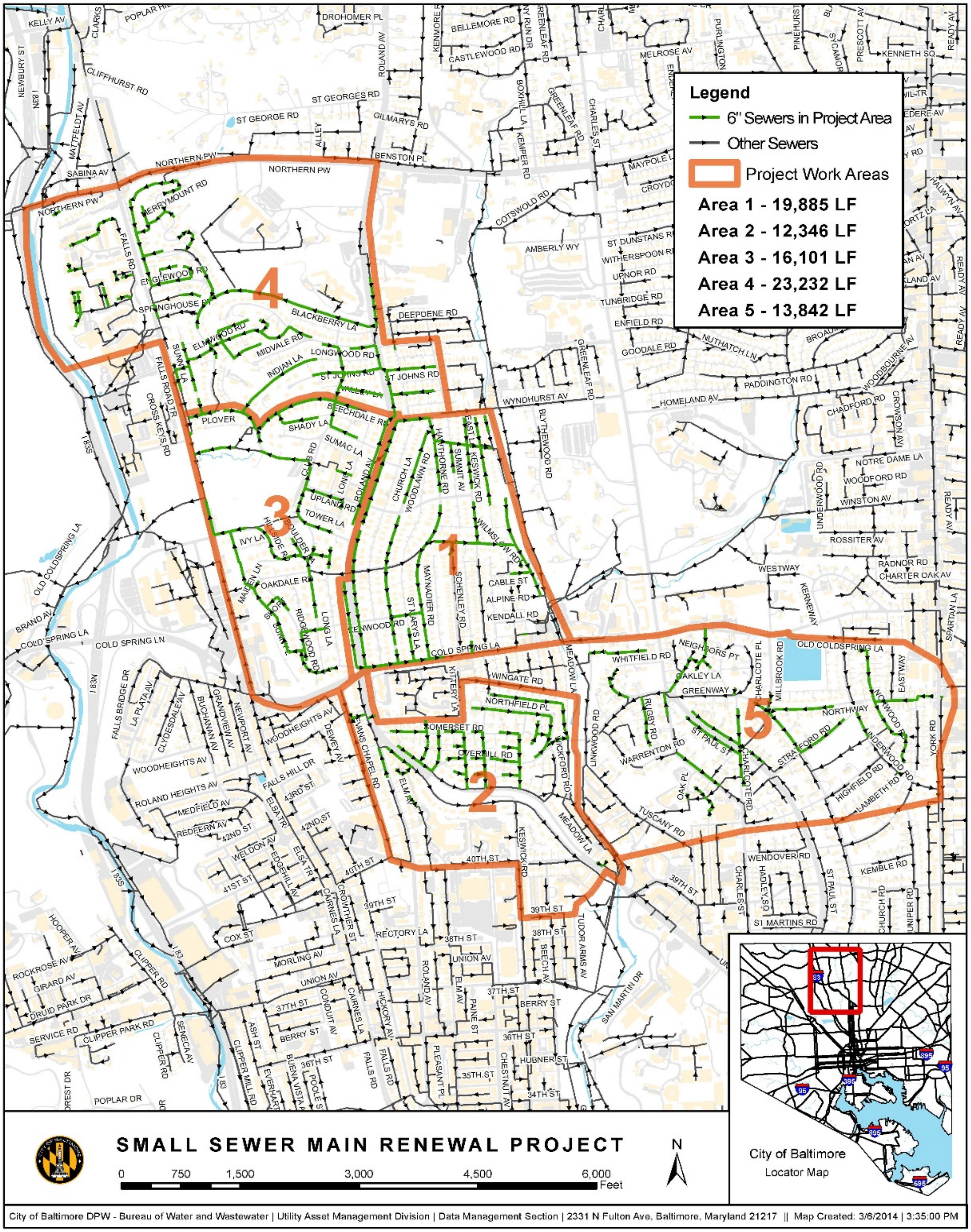

Figure 3: Small Mains Project Areas

INSPECTION APPROACH

Sewer mains 6-inches in diameter are not common and often difficult to clean and inspect. Cleaning and inspection equipment is available to perform work in 6-inch sewers, but the complex geometry of the system in the project area is expected to result in incomplete inspections. As previously described, the sewer segments in the project area are short – three feet long - and often deflected or "crimped" to negotiate bends and changes in grade. Additionally, there are a limited number of manholes to access the pipes. In 2011, approximately 2.9 miles of the 6-inch sewer located in the project area was inspected in a pilot study under the Roland Park Sanitary Sewer Investigation. The investigation revealed that approximately 20% of the inspections were incomplete due to roots, sags, sewer geometry, and inadequate access.

Where the traditional CCTV equipment is unable to inspect the small sewers due to the sewer geometry, a push camera will be utilized. A push camera consists of a small camera on the end of a flexible rod that is capable of navigating the bends and elevation changes in the small sewers. Push cameras are able to collect quality data recorded in PACP certified software, much like the traditional CCTV crawler. Available features include color video, color photos, pan and tilt, footage tracker, and text writer. These features allow the data to be recorded per the PACP.

Figure 4: Typical push camera

As mentioned above, the system does not have an adequate number of manholes required to access the sewers for cleaning and inspection. Historically, in order to save construction cost, lampholes were installed in place of manholes throughout the 6-inch sewer network in the project area. A lamphole is a narrow shaft opening, typically the same diameter as the sewer main it intersects, used for lowering a light and mirror to see if the sewer is blocked. This method of condition assessment does not provide detailed information about the condition of the pipe or reason for pipeline obstruction. Where conditions are found that prevent completion of the condition assessment and where a history of SSOs or basement back-ups exist, the installation of a manhole will be evaluated to facilitate the inspections. Manholes in conformance with current standards will be installed at locations deemed necessary.

RENEWAL RECOMMENDATIONS

The detailed condition assessment performed in Phase I will provide the engineer with information about the structural and operational defects. The engineer evaluates the effects of these defects on the ability of the sewer to covey sewage and makes recommendations for renewal. Renewal decisions are made holistically throughout the entire project area to avoid making numerous localized point repairs on a line that should be replaced in its entirety.

There are two recommended methods of renewal: open trench repairs and/or the use of trenchless technology. Open trench repairs include point repairs, typically at least 12 feet in length, and pipe line replacement. If the pipes are recommended for replacement, the 6-inch main will be replaced with an 8-inch main to conform to the standards. In addition, manholes will be installed as needed to increase accessibility to the system.

Trenchless renewal options include pipe bursting and cured-in-place pipe (CIPP). Pipe bursting is another renewal option that will result in upsizing of the pipe to 8-inches in diameter to conform to City standards. Pipe bursting requires only a small excavation footprint for the equipment and entry for the new pipe. The new pipe is pulled through the existing main, which is fractured or displaced in the process. CIPP is a structural liner that is installed on the interior of the existing pipe. CIPP will be utilized on pipes where there is a reduction in capacity due to root intrusion, and the protrusion of structural defects is minimal. The installation typically extends from manhole to manhole along the full length of the main. In cases where a full length CIPP cannot be installed and a localized defect requires a trenchless repair, a sectional CIPP will be used.

Figure 5: Open Trench Point Repair


Figure 6: Pipe Bursting Equipment

INSPECTION RESULTS

As the PACP-coded CCTV has been submitted by the Contractor, engineers have been reviewing the findings and making renewal recommendations. The four ratings provided – Overall Structural, Overall O&M, Structural Quick and O&M Quick – were all evaluated to determine the appropriate type of renewal action. For example, if the overall structural rating is high, indicating that there are several defects along the pipe segment, a manhole to manhole renewal is recommended. Conversely, if it is low, indicating an isolated defect, a point repair is recommended. The O&M ratings document the presence of roots, which is typically addressed by lining the pipe segment.

As of the time this paper was written, the condition assessment of three of the five inspection areas has been completed. Table 1 summarizes the status of the inspection and repairs:

Table 1: Project Status Summary

Completed inspections (100%± CCTV obtained; accepted submittal)	37,000 ft.
Rejected CCTV footage (e.g. video quality)	1,200 ft.
Incomplete survey (i.e. abandoned survey; urgent repair may be warranted)	8,800 ft.
Upsizing/Pipe Bursting	7,000 ft.
Number of Point Repairs	40
Pipe replacement	600 ft.
CIPP	1,500 ft.
Redesign (i.e. requires engineering to design new pipe layout)	2,300 ft.
No Action (i.e. pipe in good condition, no history of work orders, SSOs, or basement backups)	30,000 ft.

Some of the work along a 1.3 mile stretch of roadway was performed on an urgent needs basis to avoid conflict with an upcoming street-scaping project. In addition, long runs of 6-inch mains that are in wooded areas along streams have been referred to the Trunk Walk Program. Under this program, the entire reach of pipe will not be inspected; rather, the manholes will be inspected in accordance with MACP and the mains will be dye tested at stream crossings to ensure the pipe is structurally sound. Since there are very few laterals off these mains, the risk of service interruption to customers is low.

LESSONS LEARNED

Although this project is on-going, there have been some early lessons learned:

1. Inspecting 6-inch mains is a cumbersom task that requires an experienced CCTV crew. The CCTV camera gets stuck easily on protruding taps and sharp bends so understanding the equipments limitations is important. Even with an experienced crew, it was necessary to dig up a couple cameras during the course of this project.
2. The pipe geometry, not the structural integrity of the pipe, is often the root cause of historic SSOs and/or basement/back-ups. In 8-inch pipes and greater, typically there is a structural defect in the pipe or other blockage due

to roots or grease that is the root cause of a known SSO or basement back-up. However, we are finding that this is not always the case in the 6-inch mains. The crimped pipe catches debris, which accumulates and eventually causes a blockage in the pipe. Since the pipe segments are only 3 feet in length, there is a lot of opportunity for blockages that result in disruption of service.

3. Defects are relative. In other words, a small defect that would not disrupt service in a larger pipe, is magnified in a 6-inch pipe and has a greater probability of causing problems. For example, an minor offset joint that would normally go unnoticed, impedes the CCTV inspection equipment and provides opportunity for debris to catch.
4. Communication with nearby schools and businees should be your first priority when beginning a pipeline condition assessment project. Early communication fosters a favorable working relationship that helps ease the burdens associated with temporary detours (pedestrian and/or vehicular), disruption of sewer use, and scheduling of community events. During this project, reasonable accomodations were made to the local school by providing an additional flagger and crossing guard to help direct the school children returning from summer vacation. Local business were accomodated by putting them on bypass so their sewer use would be unaffected during the pipe replacement project. Communicating what to expect and making small accomodations was well worth the time.

CONCLUSION

The 16 miles of small diameter mains in the Jones Falls Sewershed that were installed by a private developer at the turn of the century are reaching the end of their service life, resulting in inadequate sewer service to the residents in this area. Using CCTV technology and PACP coding, the condition assessment will enable engineers to evaluate the root cause of long-standing problems, including chokes, SSOs and basement back-ups. A holistic condition assessment will in turn lead to the design and construction of pipeline renewal. While renewing the infrastructure via open cut or trenchless technology, manholes will be added to the system to improve future inspections and cleaning efforts. At the completion of the Small Main Renewal Project in 2018, customer service complaints are expected to decrease significantly, thereby achieving the goal of the project.

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Boston Water and Sewer Commission: Data Integration to Support Asset Management

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Abstract

The Boston Water & Sewer Commission (BWSC or Commission) with the help of consultants CH2M developed a systematic and robust approach to asset management for BWSC's wastewater and storm drainage system. BWSC is moving their infrastructure assessment and rehabilitation program from reactive to proactive responsiveness. BWSC has set a goal of completing (closed-circuit television) CCTV inspections for approximately 10% of its sanitary sewer system (90 miles) each year to establish a baseline condition of their system. To achieve this, BWSC's inspection program was re-engineered. This included;

1. Implementing tools for automation, data analysis, and reporting
2. Implementing a risk-based approach to planning future inspections

The re-engineered inspection program has allowed the Commission to meets its goals related to collecting, storing, and analyzing condition assessment data. SCREAM reports enable BWSC to document trends related to maintenance and condition of sanitary and storm drain system. The major findings of the program to date include;

- Structural condition of most inspected pipes was better than anticipated.
- The age of the pipe is not a good indicator of its structural condition.
- Pre-cleaning surveys found that significant amounts of sediment had accumulated in many pipes.
- The most frequently identify remedial measures for the inspected pipes are cleaning and maintenance rather than rehabilitation or replacement.

As the Commission continues to progress through its system on an annual basis, the re-engineered inspection program and tools are expected to help the Commission focus future investments to provide the most benefit for the City of Boston.

INTRODUCTION AND BACKGROUND

Historically, inspection data was stored in various places within departments and, other than videos, was not utilized. Reviewing videos was heavily relied upon to make asset management decisions. BWSC recognized this practice would not be sustainable as the miles of pipes inspected each year increased. They set a goal to improve the practice of collecting, storing and analyzing condition assessment data.

It was also difficult for BWSC to plan work because of the numerous sources and variable data quality. Key fields such as pipe identification (ID) and manhole IDs were rarely entered accurately into the inspection. Without knowing where the CCTV crews had been made it difficult to confidently issue work orders for proposed inspection work.

To address the data management and planning challenges, CH2M implemented the condition assessment tool System Condition Risk Enhanced Assessment Model (SCREAM). SCREAM is a data collection and analysis tool that collects inspection data, scores the inspections, and provides several reports. It provides standards and quality control before it enters the system. SCREAM centralizes all the inspection data and media into one location. Therefore, the data is more easily analyzed and SCREAM's scoring algorithm provides the most accurate inspection scores. Reports are available via the internet to all employees at BWSC. A risk-based approach was implemented in conjunction with SCREAM to help the Commission prioritize their system for inspections.

OBJECTIVES AND PROBLEMS

CH2M conducted a review of existing work practices and a review of five (5) years of historical CCTV inspection data. Historically, a large number of entities at the Commission have been engaged in performing and/or contracting work related to inspections of the Commission's assets. This has included inspections performed for operations, engineering, planning, construction and regulatory compliance. The methods used for these inspections and the management of data related to the inspections has varied significantly. The review produced the following conclusions;

- Many inspections are recorded using their computerized maintenance management system (CMMS), but not all inspection results have been maintained in an electronic format. This has made it difficult to include historic data for long term (predictive) analyses.
- After some adjustments were made to the data collected over the last 5 years, 53% of the available pipe inspections records were linked to BWSC's geographical information system (GIS) and mapped. These inspection records only totaled 12.19 miles (0.8%) of the Commission's sanitary and storm drain system.
- Review of data for CCTV inspections performed prior to 2013 indicates that many defects were documented in comment fields, and not coded per Pipeline Assessment and Certification Program (PACP) standards. This limited the value of this data toward characterizing pipe condition and for identify appropriate

mitigation measures. Correspondingly, the Commission has placed a higher value on CCTV videos than the PACP databases.

- Approximately 3% (29 miles) of the sanitary system is inspected per year, which is below a common industry target of approximately 10%

Based on the conclusions, it was determined that BWSC's CCTV inspection program had to be re-engineered for two reasons:

1. To efficiently manage the significant increase in inspection data as the Commission increases their inspection rate to 10% (~90 miles) of their sanitary system per year.
2. To effectively utilize the inspection data for capital improvement and planning purposes.

RE-ENGINEERED INSPECTION PROGRAM

To address the data management and planning challenges, CH2M implemented the condition assessment tool, SCREAM. SCREAM is an industry-standard tool for condition assessments and analytical/asset condition scoring and is being used by multiple utilities primarily in North America. An introductory discussion and case study application of SCREAM is included in the Environmental Protection Agency's (EPA) April 2010 publication titled, *Innovative Internal Camera Inspection and Data Management for Effective Condition Assessment of Collection Systems*. SCREAM is composed of three major components;

1. A centralized database that allows the Commission to store detailed condition assessment data in one place and rapidly access that data through the use of browser-based reports. The centralized database is integrated with their CCTV inspection program to provide a seamless transition from raw inspection data to scored data for analysis using SCREAM's robust scoring algorithm. Media such as videos and photos are also incorporated into the reports.
2. A second centralized database that stores the detailed risk-based approach assessment data which is integrated with the SCREAM condition assessment database and the Commission's GIS for capital improvement and inspection planning purposes.
3. A customizable and comprehensive defect coding system that can be used to record defects and severities that match what the CCTV operator sees and that includes information type codes that captures inspection information that has value that might otherwise get overlooked. For instance, informational codes are available about issues up in service laterals that might be worth collecting such as grease deposits.

The SCREAM components become the backbone of the re-engineered CCTV inspection program and workflow;

1. Planning

2. Inspecting
3. Analyzing

1. PLANNING

The planning process utilizes a risk-based approach which was conducted for the Commission's entire sanitary and storm drain system (~1,500 miles of pipe). Risk scores are the product of the likelihood (LOF) and consequences of failure (COF) score developed for each pipe segment. Both the likelihood and consequence risk components address the properties and conditions that are necessary to meet the Commission's service goals and the potential impacts to the public or environment if the asset were to fail. Factors contributing to LOF and COF for sanitary and combined pipe are shown in Table 1.

Table 1. Risk Factors for Sanitary & Combined Pipe

Likelihood of Failure (LOF)	Consequence of Failure (COF)
Physical Condition (observed or predicted)	Financial (cost of repair/replacement)
Maintenance (observed or history)	Public Health and Regulatory
Wet Weather Performance	Proximity to Critical Customers
Sediment Build-up	Public Image
Corrosion	Environmental Impacts

The risk-based approach is a combination of a Top Down and Bottom Up approach. The Top Down approach uses decay curves for different pipe materials to estimate the physical condition based on the pipe's age while the Bottom Up approach uses internal pipe CCTV inspection results to produce structural and maintenance scores. Bottom Up information is stored in the condition assessment database and is used in lieu of Top Down information because Bottom Up inspections provide more detailed and accurate account of the pipe's condition. The Top Down information is stored in the risk assessment database. A live connection is created between the risk assessment and condition assessment databases and provided real-time risk updates after inspections are completed and scored. A live connection is also created between the risk assessment database and GIS (Figure 1) to allow the Commission to use GIS for inspection planning purposes.

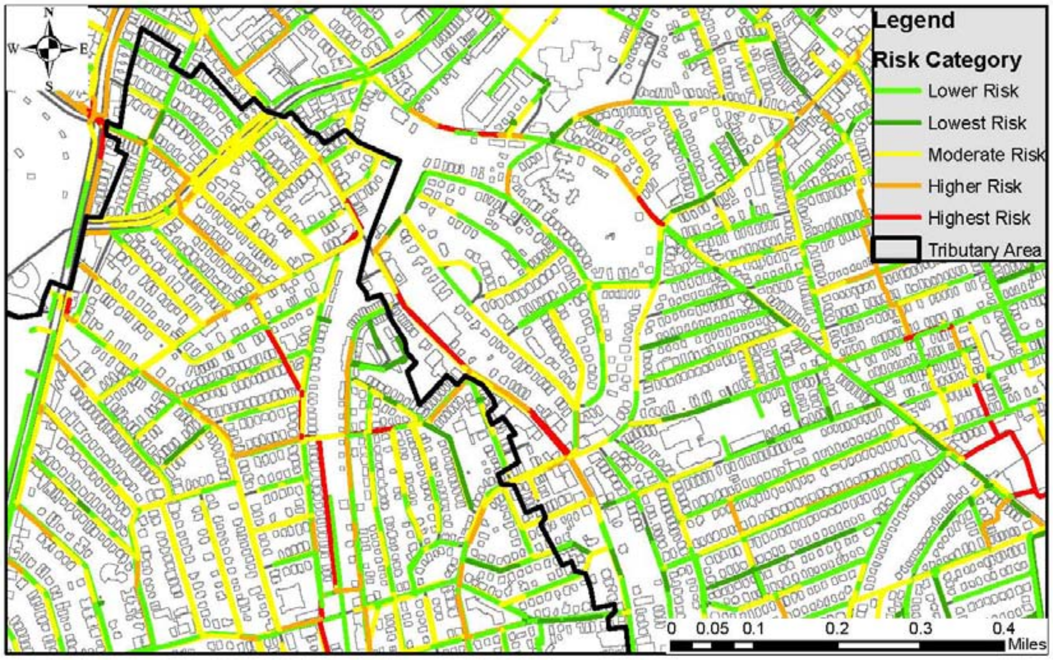

Figure 1. GIS Map of Risk-Based Planning

2. INSPECTING

Using the pipes selected from the planning process, the Commission or a contractor performed the CCTV inspections. The Commission previously used the CCTV inspection process as described by the National Association of Sewer Service Companies (NASSCO) however, this was changed primarily in two areas:

1. SCREAM defect codes were used instead of NASSCO's PACP defect codes. The Commission's and contractor's CCTV software was configured with the SCREAM defect code set.
2. The amount of debris in the pipe prior to cleaning was recorded. Over time, this information can be used to approximate the rate of debris build up which in turn will refine the cleaning frequency of the pipe segment.

Upon completion, inspections are imported into the Commission's CCTV software using a quality assurance/quality control (QA/QC) SCREAM tool. The QA/QC tool provided two significant data management improvements:

1. It converts numerous CCTV software outputs into the input required for the Commission's CCTV software. This means that the Commission's contractors were not required to use the same CCTV software as the Commission. This provided flexibility for the contractors.
2. It provides comprehensive QC measures to standardize the CCTV inspection data. One of the most important QC measure was to make sure the inspection pipe ID

matched to the pipe ID in GIS. Implementing this QC measure brought the mapped percentage from 53% discovered during the review to nearly 100%. Having matching IDs is critical when analyzing the inspection data.

Once imported into the Commission's CCTV software, the inspections are transferred automatically on a nightly schedule to the SCREAM condition assessment database where they are scored and made available in the browser-based reports for the Commission to view and analyze.

3. ANALYZING

Analyzing the inspection data is key to running a successful asset management program. It identifies operational and maintenance needs and identifies capital projects. The Commission analyzes the data by using the SCREAM browser-based reports. In general the browser-based reports can be used by the Commission for several reasons;

- To quantify how many miles of pipe have been inspected in total and per contract.
  - SCREAM helps manage the 10% inspection target per year and to check progress of their subcontractors.
- To inquire about specific pipe segments and to review the inspection details and/or watch the inspection video.
  - SCREAM provides individual pipe segment reports including access to videos and photos which can be used as supporting information for the Commission's other projects such as illegal connection investigations or for operational events such as sanitary sewer overflows.
- To review high scored pipes to either place in a capital improvement project, preventative maintenance cycle, or repair major defects found.
  - SCREAM provides a quick summary of structural or maintenance needs which can either be addressed by operations or engineering by placing the pipe(s) on a preventative maintenance cycle or in a capital improvement project.
- To review the overall condition grade of the system per neighborhood (Figure 2). The condition grade can further be separated into structural and maintenance grades.
  - SCREAM provides charts depicting the condition grade breakdown of areas - in this case the neighborhoods. This information could be used to create a capital improvement project in a particular neighborhood.

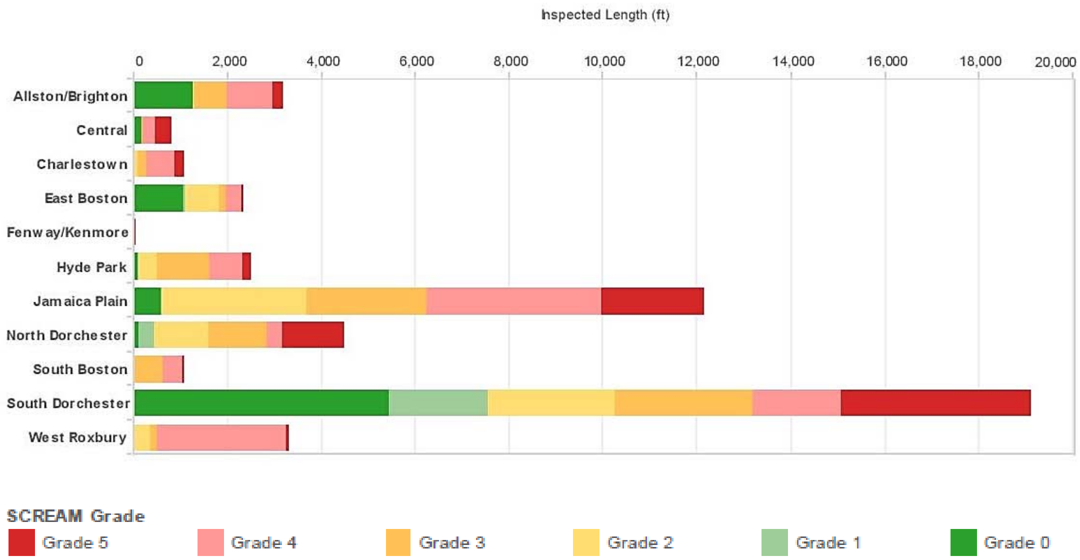

Figure 2. Total Condition Grade per Neighborhood

CONCLUSIONS & FINDINGS

The integration of the risk-based approach and the SCREAM condition assessment tool provides the Commission with the necessary tools and workflow to achieve their asset management goals. SCREAM enables BWSC to document trends related to maintenance and condition of sanitary and storm drain system. It was found during the program that the structural condition of most inspected pipes was better than anticipated (Figure 2). Only 6% of the pipes inspected fall into a grade 5 which are considered failed or near failure due to an extreme defect such as a collapse or severe break in the pipe resulting in 25-30% of the wall missing. Other structural defects like cracks and displaced joints were found; however these issues showed no indication of hindering the performance of the system.

Age is generally considered as an indicator of structural condition because it is assumed that degradation occurs over time (Figure 3). Age is used in the Top Down approach to estimate the structural condition in the absence of inspection data – a score of zero indicates good structural condition while a score of 100 indicates poor structural condition. It is expected that the results from the condition assessment, the Bottom Up approach, will result in a similar trend as the Top Down approach. During the project, different ages of pipe were selected for inspection in order to verify the Top Down approach. Out of the pipes inspected, 59% of them were 100 years old or greater and structural issues ranged from minor to major issues indicated by the SCREAM structural score (Figure 4). It can be seen that age is not a good predictor of the structural condition and that an inspection should be performed in order to determine the structural condition.

Figure 3. Top Down: Estimated Structural Condition vs Age

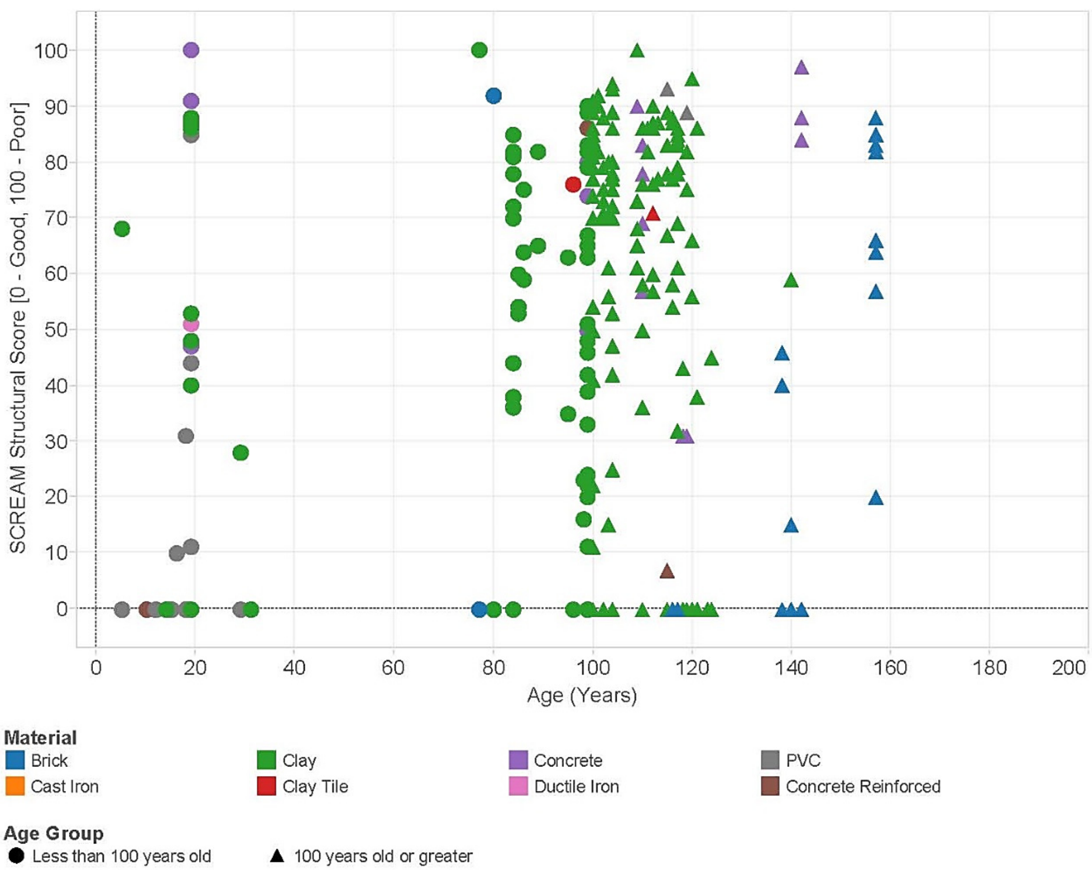

Figure 4. Bottom Up: Structural Condition vs Age of Inspected Pipe

On the other hand, maintenance issues are the most predominant issue for the inspected pipe. Pre-cleaning surveys found that significant amounts of sediment had accumulated in the pipes. 37% of the pipes required heavy cleaning before they could be inspected. Heavy cleaning is when three (3) or more passes are made with the jet nozzle to remove sediment. As seen in Figure 5, several Boston neighborhoods required heavy cleaning.

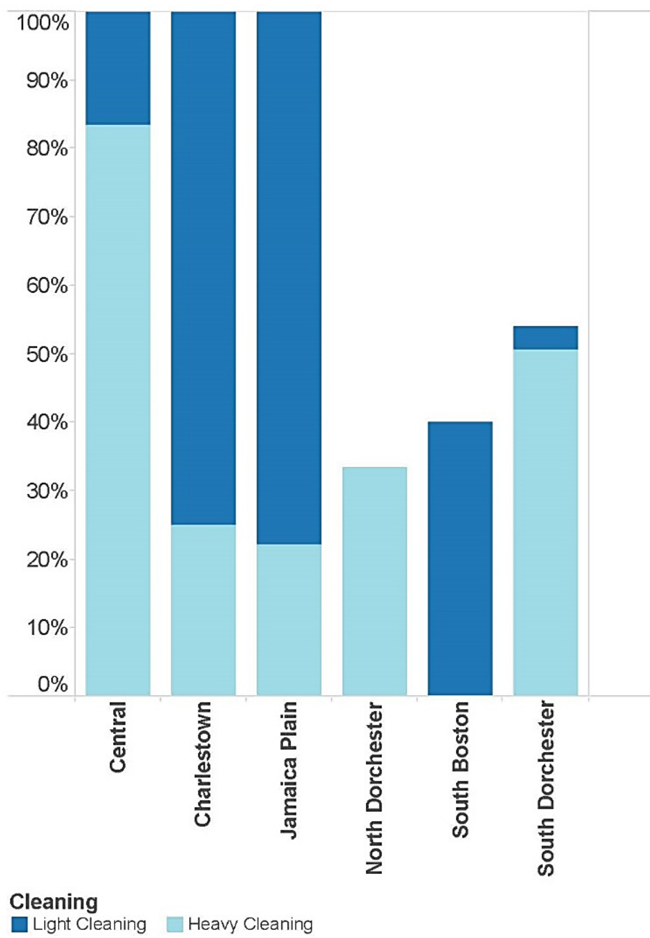

Figure 5. Cleaning Percentages of Inspections

Once the sediment was removed, the inspection was conducted. The most frequently identify remedial measures found were cleaning and maintenance rather than rehabilitation or replacement. Only two collapses were found which were repaired immediately by the Commission. Maintenance issues were more prominent than structural issues. Out of the recorded maintenance issues, 24% of them related to obstacles such as bricks and stones, deposits such as mineral deposits and encrustation, roots from trees and plants and debris that was missed during initial cleaning (Figure 6). Maintenance issues were removed to maintain system performance.

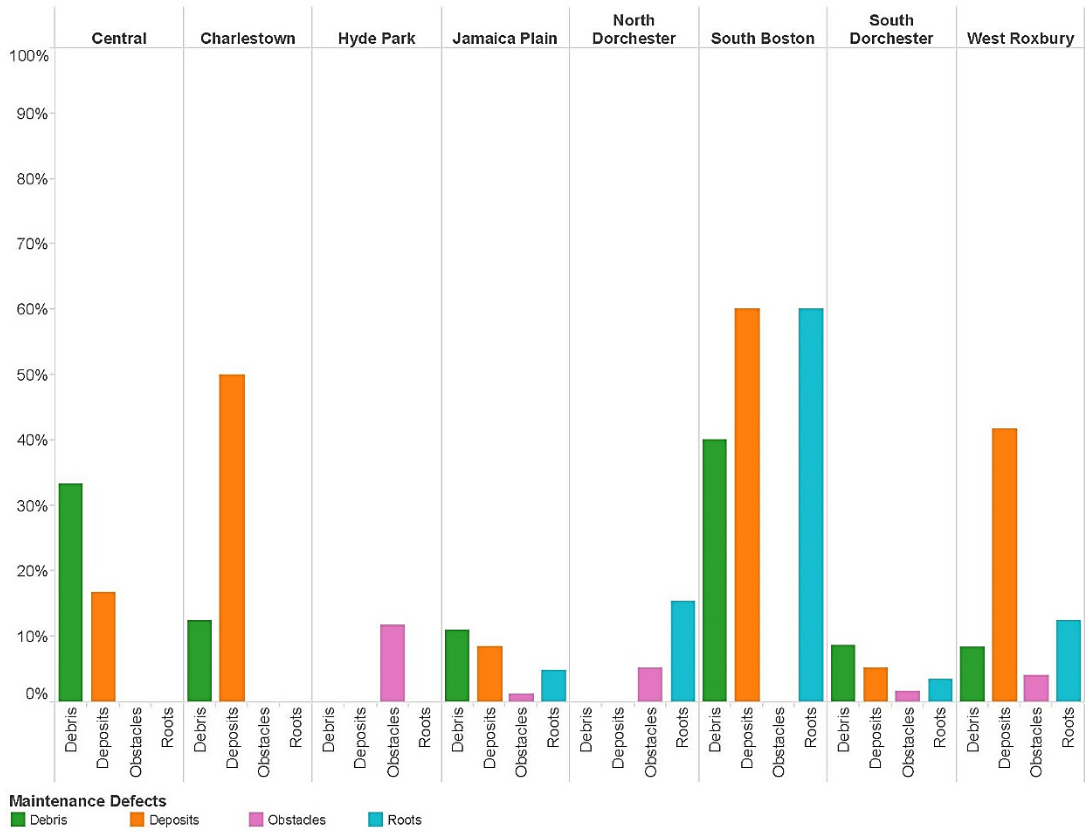

Figure 6. Cleaning and Maintenance Issues

As the Commission continues to progress through its system on an annual basis, the re-engineered inspection programed will allow the Commission to work towards achieving their asset management goals and accomplish their long-term objectives;

- Measuring the degradation rate of assets.
- Measuring the effectiveness of maintenance strategies.
- More accurately predicting the remaining life of its assets and plan for their replacement.
- Avoiding interruption of service caused by asset failures.
- Focusing on proactive management of its assets rather than reactive activities.
- Maintaining desired levels of service at the lowest life cycle costs with acceptable levels of risk.
- Reducing the overall risk of the sanitary sewer and storm drain system.

This new process will help the Boston Water and Sewer Commission focus future investments to provide the most benefit for the City of Boston.

ACKNOWLEDGEMENTS

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Pipeline Asset Integration Planning for a Major Water Supply System: The Southern Delivery System, Colorado Springs, CO

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Abstract

The Southern Delivery System (SDS) is a regional project that will bring water from the Arkansas River to the City of Colorado Springs, the City of Fountain, Security Water District and Pueblo West Metropolitan District. Core components of the project consist of connection to the Pueblo Dam, 45-mi (72-km) of 66-inch (1.7-m) diameter raw water pipeline, three pump stations with total connected horsepower (hp) of 26,750 (19,947 kw), 50 MGD (189 ML/D) water treatment plant and finished water pump station, 4 miles (6-km) of finished water pipeline, and environmental mitigation to meet regulatory requirements. Although integration of SDS is part of a comprehensive integration plan, the primary emphasis of the paper is on the raw water pumping/pipeline operational strategies, including:

- Integration planning
- Asset hierarchy
- Commissioning/startup
- Operator training
- Warranty
- Optimization
- QR technology

Through the use of these integration strategies, SDS will deliver water to customers in the spring of 2016.

INTRODUCTION

Currently, raw water delivery to Colorado Springs is accomplished through an extensive system of diversions, reservoirs, pipelines, and pumping stations as shown in Figure 1. Raw water is currently delivered to the following four water treatment plants (WTPs):

- Pine Valley WTP – northwest region of the Colorado Springs Utilities (Utilities) service area.
- McCullough WTP – located adjacent to the Pine Valley WTP.
- Mesa WTP –foothills north and west of downtown Colorado Springs.
- Fountain Valley Authority (FVA) WTP –Utilities' service area

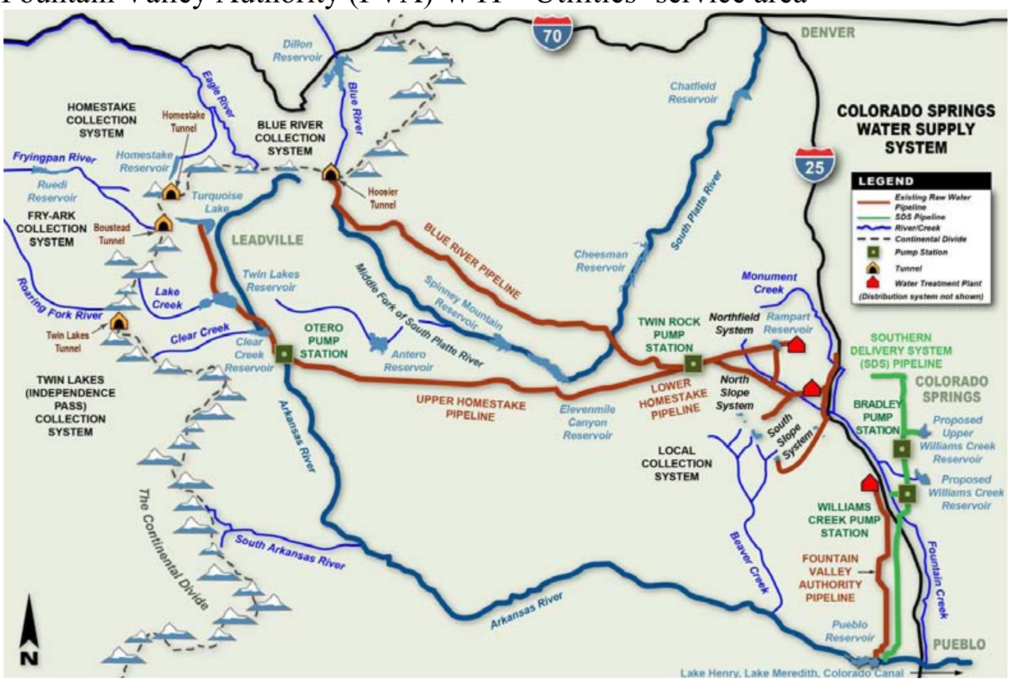

Figure 1. Colorado Springs Raw Water Supply System

Southern Delivery System

The SDS is a regional project in Colorado that will bring water from the Arkansas River to residents and businesses in the City of Colorado Springs, the City of Fountain, Security Water District and Pueblo West Metropolitan District, known as the Participants. Colorado Springs needs SDS to help protect the community against drought, to provide water for the growing population, and to provide water system redundancy. Core components of the project consist of connection to the Pueblo Dam, 45-mi (72-km) of 66-inch (1.7m) diameter raw water pipeline, three pump stations with total of 26,750 hp (19,947 kw), 50 MGD (189 ML/D) water treatment plant and finished water pump station, 4 miles (6-km) of finished water pipeline, and environmental mitigation to meet regulatory requirements.

In July 2009, the Colorado Springs Utilities Board (Board) authorized construction of SDS, setting an in-service date of 2016. The authorization was preceded by a six-year permitting process culminating in receipt of a Record of Decision *(U.S. Bureau of Reclamation, 2009)* from the U.S. Bureau of Reclamation, as well as authorization from neighboring Pueblo County under the State of Colorado 1041 permit process.

The completion date of 2016 was chosen to allow the Board an orderly and systematic series of rate increases necessary to finance the project. Because determination of probable construction costs were critical, the owner and program manager requested the design engineer undertake systematic value engineering of the water conveyance pipeline. This was essential, as the SDS completion schedule was contingent on early construction of several pipeline segments.

Purpose and Need

The purpose of the project is to provide a safe, reliable and sustainable water supply for the Participants through the foreseeable future. The Participants have three needs that SDS fulfills 1) develop water supplies to meet future demands through 2046, 2) develop additional water storage, delivery, and treatment capacity to provide system redundancy, 3) perfect and deliver the Participants' existing Arkansas River Basin water rights. The SDS, as depicted in Figure 2, was chosen to provide a redundant method of delivery for Colorado Springs western slope water supply and ensure capacity for population growth in the region.

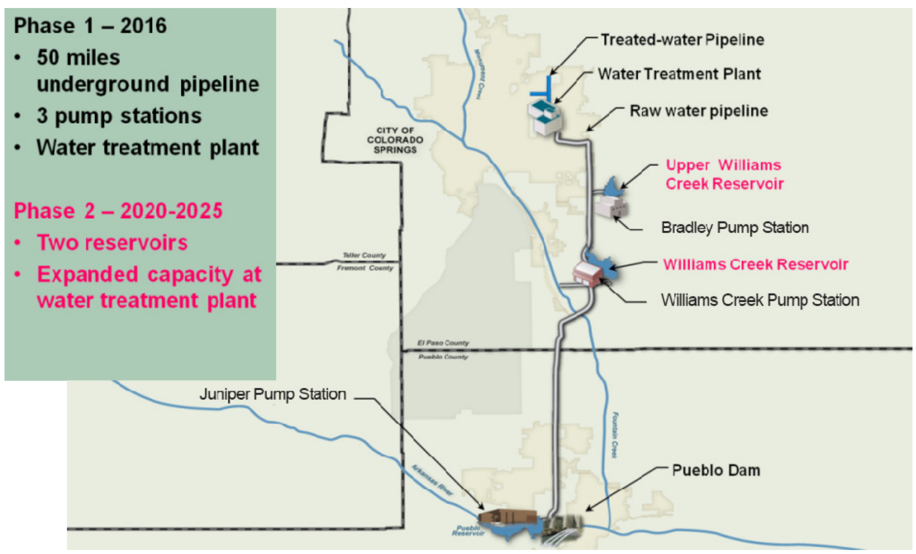

Figure 2. Southern Delivery System

INTEGRATION PLANNING

The SDS integration process is defined as the seamless transition from construction through commissioning and startup, to integrated daily operation of Utilities' existing water supply systems and supporting enterprise systems and tools. The integration plan provides an operationally efficient and time critical transition that returns the best value to project stakeholders and to Utilities' customers. This paper describes the strategic planning strategies, practices, and benefits of early development of a comprehensive integration plan for the SDS pipeline to demonstrate that, through the use of these and other water system integration strategies and practices, SDS will be online and delivering water to customers in early 2016. It will also provide SDS operations staff with essential data, tools, and training necessary to facilitate a smooth and effective transition from construction, testing, and commissioning to efficient operation.

Integration of SDS began during the early stages of planning and implementation of the program in 2011. In April, 2013, the SDS program leadership recognized the need for a centralized, comprehensively scoped, scheduled, and budgeted effort to ensure a successful transition from construction to integrated operations. Since that time, integration efforts have been focused on progressing time critical activities and developing a comprehensive scope and corresponding organizational structure to enable effective management of SDS integration activities. Based on these scoping and organizational efforts, the following four primary integration elements were developed and will be addressed in this paper as follows: 1) commissioning and startup, 2) operational integration 3) a summary of technologies, tools, and their integration with various Utilities' enterprise systems, and 4) optimization of constructed facilities.

Commissioning and Startup

The objective of commissioning and startup is the safe, logical, and systematic testing, verification, and documentation that all aspects of the equipment, components, systems, and facilities satisfy the functional and performance requirements of SDS as depicted in Figure 3. When commissioning and startup is complete for the pipeline and raw water pump stations (RWPS), those facilities will be subject to a seven day performance test and upon satisfactory completion per the specifications, will be complete to the level that Utilities can use those facilities as intended. The methodology for implementing commissioning and startup at the RWPS and WTP will be as defined in the relevant contract documents and startup plans.

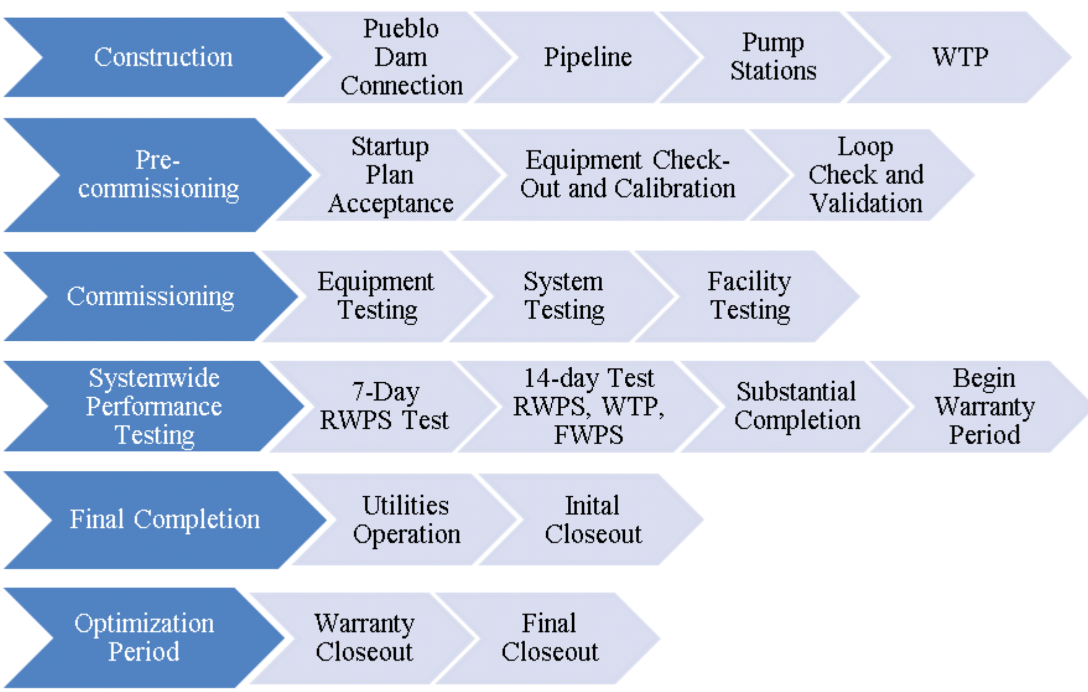

Figure 3. Commissioning and Startup Workflow Diagram

Startup Database

The SDS team developed a hierarchical database to capture asset management systems and refine the commissioning and startup database structure and naming convention for consistency and integration into Utilities' enterprise asset management system to achieve duplicate structure, easy access to data, confirm regulatory compliance and develop water consumption data.

A key focus in the integration of assets into the utility is that the required asset information matches the business needs and is transitioned into the business systems, and that the asset owners and operators have full knowledge of the assets they have acquired. Components of the pipeline asset hierarchy include equipment tagging, asset ID, location identification (LID) and GIS interface.

Utilities uses Maximo and GIS to manage assets throughout the Enterprise. In anticipation of the asset handover process, SDS assets have been loaded into these systems and will include assets scheduled maintenance information, which will in turn support the long term management of the assets through their life cycle.

Operations and Maintenance Training

The SDS team developed operator training course material and facilitated a series of pipeline operator training classes. In addition to water conveyance and safe operation of the new pipeline, other topics include filling and draining to important access procedures unique to accessing the remote SDS pipeline.

Access

The raw water pipeline spans approximately 45 miles (72-km) passing through a mixture of private, federal, state, and local government owned and controlled land including the Bureau of Reclamation, Colorado Parks and Wildlife, Pueblo West, Pueblo County, El Paso County, and the City of Colorado Springs. There are several major utility crossings where the raw water pipeline crosses under the major infrastructure. The raw water pipeline resides within permanent easements, right-of-ways, open range land, and single family residential dwellings. The raw water pipeline shares these easements and right-of-ways with a number of other utilities. This sharing of easements is generally in the southern section of the raw water pipeline.

Due to the wide range of complexity in the land agreements (such as regulators, local government, and property owners), access to the raw water pipeline and associated appurtenances needs special attention. When accessing the raw water pipeline, Utilities' staff give consideration to the following:

- Agreements with landowners that may require advanced notification to landowners, businesses, the County, etc. prior to accessing the pipeline and appurtenances
- Environmental conditions, such as revegetation and the prevention of the spread of weeds
- Boundary of easements both construction and permanent
- Crossing and working in the vicinity of other existing utilities (weight restrictions when crossing existing utilities)

Warranty

Managing the warranty of any new infrastructure is vitally important to the successful delivery, handover, and transition to operations. The SDS team has developed a warranty tracking database to track deficient work or defective equipment. Prompt and effective resolution of issues encountered during the defect correction and warranty period is critical to ensure long-term performance and reliability of SDS infrastructure. The following are tracked during this period:

- Asset failure/repairs including details on failure mode
- Cost or impact of asset failure
- Time to restore asset back into service
- Sign-off and acceptance that the defect has been rectified
- Frequency of failures of assets or groups of assets
- Non-performance of assets against agreed operational performance standards and the basis of non-performance
- Frequency or duration of asset non-performance
- Sign-off and acceptance that non-performance to required standards has been corrected or accepted and the asset is compliant with Utilities' requirements
- Compliance with inspection and service requirements as detailed in suppliers specification
- Any changes to assets that involve full replacement will need the new asset attribute data and installation details recorded for incorporation into the asset register

The SDS team developed a system to track and monitor SDS facilities and equipment during the defect identification period with a high level of detail and transparency to how they have been managed to allow appropriate resolution of any warranty issues between Utilities, construction contractors, and suppliers. Maximo is currently being configured to track and manage assets during the warranty and defects period, and it will be important that the performances of the assets are reflected in Maximo as an input into the long term service life management of the asset. Figure 4 indicates the warranty process workflow.

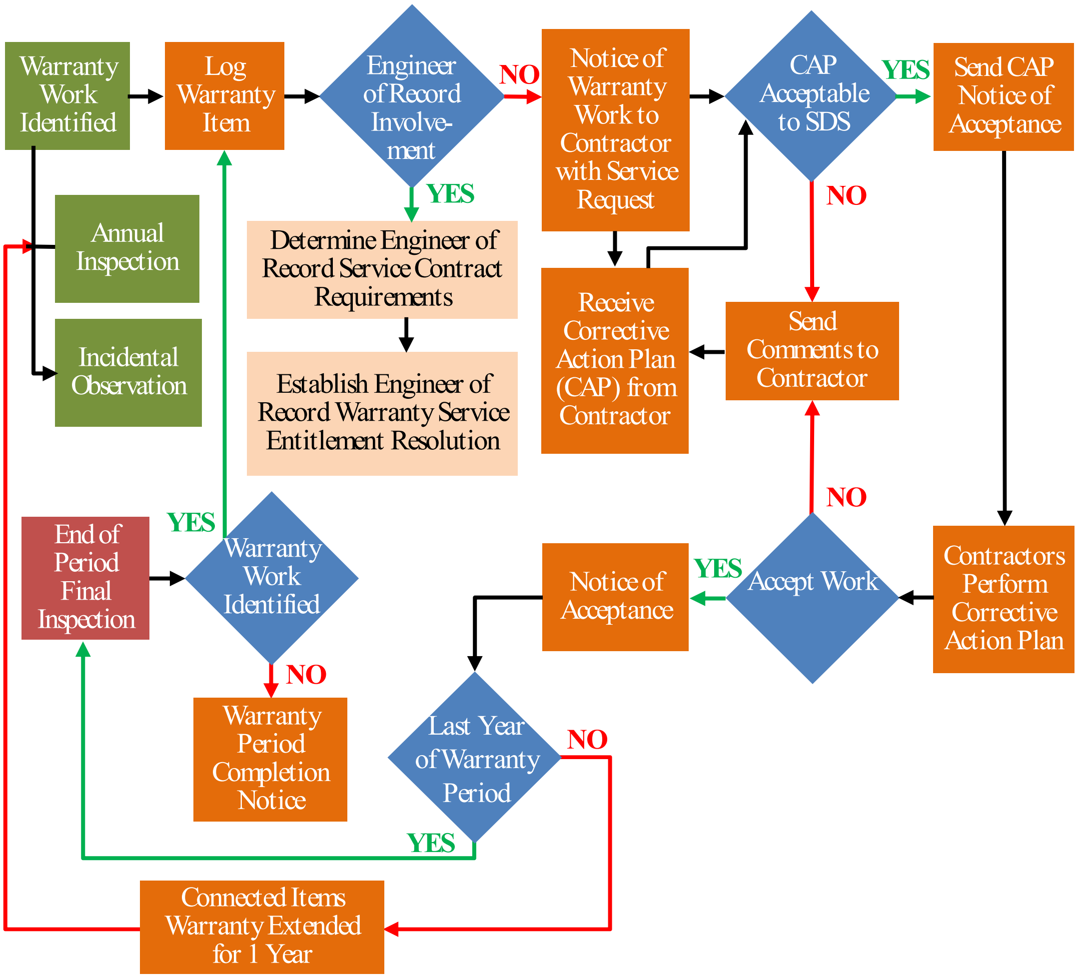

Figure 4. Warranty Process for Workflow Diagram

OPERATIONAL INTEGRATION

Operational integration planning is essential to establish the basis of operation and integration into Utilities' existing water operations *(Colorado Springs Utilities, 2014)*. Once in service, the SDS will be a new major water supply adding to the Utilities' existing water supply system providing reliable water infrastructure to meet the needs of future generations. The SDS integration team, Utilities' stakeholders, primarily from the Water Services Division, along with many functional groups established the following operational scenarios.

Normal Operational Scenario(s) are those mode(s) of operation that are regular and foreseen and are implemented initially and in the near-term.

Unexpected Operational Scenario(s) are those mode(s) of operation that are unlikely to arise but would require a variation from normal SDS operations, requiring some type of operational shift by Utilities. Operational changes would likely include the mobilization of additional staff and infrastructure (i.e., temporary pumps), adjustments to supply zone control valve settings, adjustments to tank level operations, and pump station operations.

The SDS Basis of Operation Plan establishes the operation parameters for SDS. The Basis of Operation Plan and related technical memoranda evaluated two operation scenarios as follows:

Scenario 1 represents the least cost supply solution but the greatest water source variation to finished water customers and involves operating the SDS WTP at a constant 5 MGD (19 ML/D) throughout the year. The flow rate for Scenario 1 was selected because: (1) finished water produced by SDS is more expensive than existing water treatment plants and (2) as designed the minimum finished water production rate from SDS WTP is 5 MGD (19 ML/D).

Scenario 2 represents the greatest cost solution but also the smallest water source variation to finished water customers and involves operating SDS at a variable finished water production proportionally to seasonal finished system demands to hold the SDS zone of influence. The variable flow rates for Scenario 2 were established to mimic the fluctuations in seasonal finished water system demands. The typical winter finished water demand is 40 MGD (151 ML/D) as compared to a summer demand of 140 MGD (530 ML/D) representing a ratio from winter to summer of approximately 1:3.5 roughly matches the proposed 5 to 20 MGD (19 to 76 ML/D) finished water production rates for Scenarios 2. By matching the SDS finished water production to the system demands, the zone of influence of SDS water will be held consistent.

The selected finished water production rate for SDS with consideration given to cost and system wide redundancy/availability was established at a constant 5 MGD (19 ML/D) year round. This operational scenario is valid until either the finished water system demands require SDS to produce more than 5 MGD (19 ML/D) to satisfy demand or the occurrence of an unexpected operational scenario(s).

With the WTP finished water production rate set to 5 MGD (19/ML/D), the raw water system will need to be operated to deliver sufficient raw water to the WTP. The RWPS have been designed to deliver a maximum flow rate of 50 MGD (189 ML/D), therefore the RWPS could run for as little as 2.4 hours per day. However, it is anticipated that the raw water pumps will pump at a flow rate that maximizes pump efficiency and therefore optimizes energy cost associated with pumping.

Defining "Normal Operational Scenario(s)" is to establish the finished water distribution strategy. This work is presently under development by the SDS integration team, and has and will examine and determine the most appropriate strategy or strategies for delivering SDS water into the Utilities finished water system. To ascertain the mode of operation with respect to the finished water system, the investigation will consider several key success factors such as optimizing cost, water rights, minimizing operational complexities, and maintaining water quality.

Technology and Tools

The SDS is a combination of more than 50 miles (81 km) of large diameter raw water pipeline, three RWPS and a WTP. All of the assets are new to Utilities operations staff and the treatment plant in particular contains infrastructure (chemicals and supporting processes) not previously used by Utilities. The entire suite of assets will require a combination of planned and unplanned maintenance throughout the life of the assets, beginning at the start of commissioning. In fact, commissioning is critically important as there will need to be a clear, supporting business processes for managing and resolving defects.

In addition to the asset and maintenance information, there is a range of other information that is relevant to the SDS. For example, there are hundreds of individual land access permits, property agreements, general permits, and property and safety information that must be maintained by Utilities and easily accessed by field teams.

Utilities has recognized that the volume of information associated with the SDS, as well as its geographic extent (from Pueblo Reservoir to the WTP) of maintenance activities means that new systems and processes need to be implemented to enable the field teams to operate efficiently. The technologies, tools and processes must be supported with technical solutions that allow field teams to have accurate and up-to-date asset, and land access information in the field. Successful delivery and operational integration relies heavily on the application of several tools and adaptation platforms for use in integrating into Utilities' existing enterprise system. Table 1 is a summary of some of the tools used in the operation of SDS.

Table 1. SDS Operational Systems/Tools

Function	Benefit
Mobile Solution Tool	
• A mobile, graphical user interface with Utilities GIS map features and task bar to include Maximo asset/work order detail in a disconnected environment • GIS based land, easement access requirement • QR code reader capability and type-in lookup of assets and work orders • Latitude and longitude values for selected assets • Selection of predefined failure codes • Document retrieval from repositories such as Active Manuals and eO&M • Redlining • Access to Utilities' Automated Vehicle Location system • Mobile dispatch	• Efficient processing of work assignments down to the crew and individual • Reliable collection of field data capabilities (in support of source systems Maximo, GIS, cathodic protection and PI) • Process workflows (task progression) • Work re-assignment capability • User friendly "Check the box" process for creation and processing of work orders • Effective scheduling of work with the capability of downloading scheduled work orders
Water Management Tool *Water Management Tool (J. Edward Barnhurst, P.E., MASCE, Jack Myers, J. Russell Snow, P.E., MASCE, 2014).*	
• Evaluates and tracks water quantities needed to support filling, draining • Manages transfer of water from one constructed pipeline section to another • Manages water during planned maintenance activities • Linked to the SDS electronic O&M manual	• Management and sustained reuse of construction test water • Saved over 30 MG of water during construction • Estimates and tracks fill rates and fill volume • Estimates discharge volumes and rates for to support maintenance dewatering operations

eO&M Manual	
• Disconnected access to manuals, As-Builts, and maintenance procedures • Fast deployment • Intuitive user interface design • Leverage of existing IT infrastructure • Pure web browser application (no plugins required) • Full text search on contents and files	• Company realizes return of investment much faster • Staff can be very productive in creating manuals and keeping them updated in a short time • Company saves on initial investment cost. There is no on-going maintenance fee and minimum IT support required
Commissioning and Startup Database	
• Equipment tagging • Asset ID • LID • GIS Interface	• Unique tag number for equipment and devices • Duplicate structure to maintain consistency. • Efficiency in documenting startup testing
eO&M Manual	
• Historian database • Time series data • Trend reporting • Alerts with Geo-links to other databases	• Native OSIsoft® PI System integration, including asset and time-series data • Excel-based design tool for rapid deployment and changes
IWLive®	
• Support proactive operation • Verification of Hydraulic model vs. SCADA • Develop rapid response strategies in real time • Analyze reaction to past events • Support operator training • Limit capital costs	• Improve system understanding • Reduce service interruptions and unexpected situations • Gain knowledge of system conditions where SCADA is unavailable • Solve water quality issues • Enhance system security • Reduce costs

OPTIMIZATION

There are many complexities of large conveyance systems such as SDS comprised of large diameter pipelines and a series of pumping stations. Optimization addresses how to make SDS facilities operate most efficiently. Technologies and tools address various enterprise systems and tools used for startup, integration, and operation of SDS. The SDS team successfully developed and implemented an approach to deliver an optimum raw water pipeline and pumping stations through an innovative procurement, contracting, and design approach based on lifecycle cost and performance based selection of major pumping equipment. The results of this approach are summarized below: 1) Performance based equipment selection through a competitive process on 30-year lifecycle, 2) selected a constructor through competitive bidding, 3) achieved lump sum price for equipment below budget. Selected pumps, motors, and drives through a competitive process that included a rigorous evaluation of technical specifications, 30-year lifecycle operating costs, and initial capital costs

The combination of pumps, motors, and drives selected for the pump stations will determine the stations' long- term energy and cost efficiency. Energy efficiency was a significant focus throughout the design process and into construction bidding. After a thorough evaluation period that considered technical specifications, 30-year life-cycle operating costs, and initial capital costs, Utilities selected the successful vendor to supply pumps, motors, and drives for the RWPS using an approach developed by MWH. This configuration of major equipment offered several advantages, including the lowest combined capital and estimated 30-year operating costs. The pre-qualified construction contractors were notified to include pumps, motors, and drives supplied by the pump manufacturers in their bids. Performance design and pricing pump, motor, and drive equipment data was tabulated in the bid form *(Steve Duling; Jay Hardison; Kirk Olds, P.E.; Matthew Schultz, P.E, Christopher Ott, P.E.; and Mark Allen, P.E, 2013).*

The goal is to maximize operational effectiveness through optimization of the SDS. Figure 5 presents the SDS pump efficiency curve for the Bradley Pump Station demonstrating the increase in efficiency from the specified value to the actual factory performance data. These data show that for each SDS pump station meets specified pump efficiency and in some cases exceeds the specified efficiency by as much as 6.5 percent above the specified efficiency.

The complete efficiency data and energy cost savings for the 30-year planning horizon is presented in Table 2.

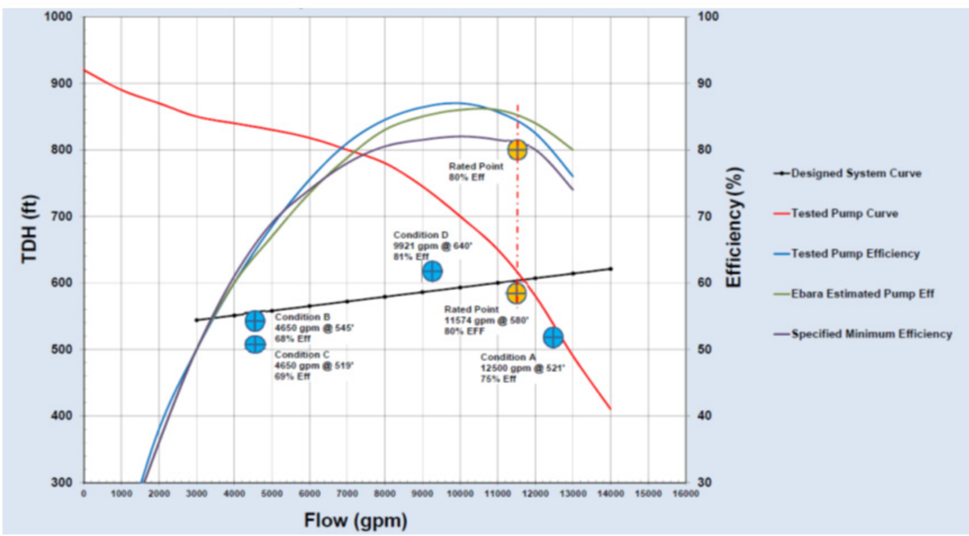

Figure 5. Bradley Pump Station Efficiency Curve

Table 2. SDS- Pump Station 30-Year Power Cost Summary

Pump Number	Date Tested	Pump Efficiency at Rated Point			Energy Impact (kWh)	Power Cost	Total Saving Each Pump Station
		Specified	Proposed	Tested			
Bradley Pump Station							
BPS-PMP1100	7/26/14	80.0%	85.5%	83.5%	174,615	$7,491	$48,939
BPS-PMP1300	8/25/14	80.0%	85.5%	83.6%	357,452	$15,335	

Pump Number	Date Tested	Pump Efficiency at Rated Point			Energy Impact (kWh)	Power Cost	Total Saving Each Pump Station
		Specified	Proposed	Tested			
BPS-PMP1400	8/29/14	80.0%	85.5%	84.3%	608,711	$26,114	
Juniper Pump Station							
JPS-PMP1100	1/9/15	80.0%	86.0%	85.6%	(772,902)	$(33,158)	$(113,467)
JPS-PMP1200	1/21/15	80.0%	86.0%	85.8%	(938,925)	$(40,280)	
JPS-PMP1600	2/2/15	80.0%	86.0%	85.4%	(368,935)	$(15,827)	
JPS-PMP7600	2/5/15	80.0%	86.0%	86.0%	(1,337,068)	$(57,360)	
Williams Creek Pump Station							
WCPS-PMP1100	9/18/14	80.0%	85.5%	85.9%	1,420,163	$60,925	$302,717
WCPS-PMP1200	9/24/14	80.0%	85.5%	86.5%	1,769,800	$75,924	
WCPS-PMP1600	10/1/14	80.0%	85.5%	86.2%	2,710,277	$116,271	
WCPS-PMP1700	10/6/14	80.0%	85.5%	85.9%	2,576,256	$110,521	

NOTES:
Pump Specified Efficiency = As-specified minimum efficiency per table 432113-1
Pump Proposed Efficiency = Proposed pump efficiency per Ebara-Technical Proposal, dated: 2/19/13, Pump Tested Efficiency = Factory-tested pump efficiency at the rated point.
Energy Impact (kWh) = Estimated power saving over 30 years operation. Power Cost Impact (kWh) = Estimated power cost saving over 30 years operation at $0.0429/kWh.

By tracking baseline and ongoing cost, the effectiveness of optimization initiatives can be closely monitored and modifications made as necessary. The SDS pump performance and associated optimization will be managed and tracked during commissioning and startup, the optimization period following substantial completion and long term over the 30-year planning horizon. One of the key aspects to effective optimization is the accumulation and subsequent analysis of performance data and to this end maximum use will be made of capturing information in OSI PI which is Utilities' enterprise data historian and is just one of the tools discussed in the previous section.

CONCLUSIONS

Various technologies and tools were implemented by the SDS team to facilitate reliable acquisition and processing of data. The mobile applications, technology tools, and enterprise systems support effective execution of maintenance activities while meeting Utilities' high standards for asset management to provide detailed documentation on all maintenance activities. Tracking activities and costs on all assets leads to predictive maintenance and away from emergency and corrective maintenance. Additionally, autonomous field crews report greater levels of efficiency in finding assets, answering their own questions, and completing work.

Based on the current factory test data for the SDS pumps and pipeline construction the life cycle cost approach used for the pumping equipment indicate a net energy cost savings of $238,189 based on the 30-year lifecycle planning horizon. This savings is in addition to the $.8.2M in savings achieved from the performance based lifecycle procurement and contracting approach used for SDS pump stations. Further testing will be conducted once the pumps are installed and tested in the fourth quarter of 2015. The pump performance-based lifecycle approach is just one of many examples of effective integration and value management, Today SDS is on schedule to be delivered in the first quarter of 2016 and currently $150M under budget This approach is a validation of sound integration planning and best value to project stakeholders by meeting one of the SDS critical success factors to build and commission best value assets that integrate with existing infrastructure and leverage the core operating talent.

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A Successful CCCP Rehabilitation on Two 96-inch CMP Culverts

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Abstract

In 2013, the City of Aurora engaged Wilson & Company Inc., Engineers and Architects (Wilson & Company) to conduct a comprehensive inspection and condition assessment of their stormwater Corrugated Metal Pipe (CMP) sewers. Approximately 41,000 LF of existing and abandoned CMP conduits, ranging in size from 12-inches to 120-inches in diameter, were evaluated and prioritized for rehabilitation or replacement. Wilson & Company completed an engineering report that included a summary of all CMP assessment findings, infrastructure evaluations, budgetary level cost estimates, priority rankings and recommendations for rehabilitation improvements. The report also specified final design recommendations for conduits that were deemed to be in eminent failure. Wilson& Company observed that a pair of 96-inch CMP pipes under a major arterial roadway, which Aurora's Public Works Department had already de-rated for traffic-loading, were severely deteriorated. Wilson was commissioned to review applicable rehabilitation techniques that had minimal public impacts. The consultant recommended Centrifugally Cast Concrete Pipe (CCCP) as the repair method that would meet all the City's objectives. This paper will discuss the development of the design for the CCCP, installation and lessons learned in the first CCCP rehabilitation project for Aurora Water.

INTRODUCTION

With an estimated population of nearly 347,000, the City of Aurora is currently the third-largest city, and one of the fastest growing communities in Colorado.

Aurora Water is a cost-of-service utility in the City of Aurora. Among its many responsibilities, is the operation and maintenance of the storm water collection and conveyance systems in the City. As part of this mandate, in 2013, the City of Aurora selected Wilson & Company to conduct a comprehensive inspection and condition assessment of their storm water CMP sewers. The inspection consisted of approximately 10 miles of deficient and aging CMP within the City.

As part of their scope, Wilson & Company developed a Condition Rating, Prioritization Ranking System, and the created a CIP Program and budget. Wilson & Company was also tasked with identifying and providing the design for the segments that required immediate rehabilitation. The twin 96-inch culverts located at the intersection of Louisiana Avenue and Biscay Street fell into the category requiring immediate attention, shown in Figure 1.

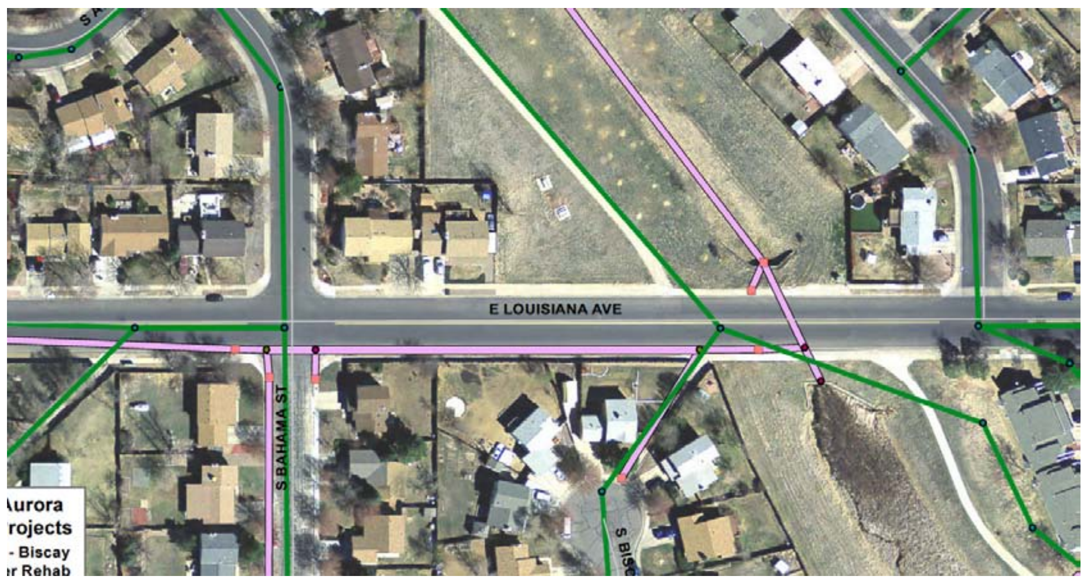

Figure 1. Street view of the project area.

The City of Aurora Public Works Department (DPW) routinely conducts its own inspection of all major culvert crossings. Their Essential Repair Findings indicated this CMP culvert was failing due to severe deterioration of the invert. As a result, the culvert was given a National Bridge Inventory (NBI) Code of 3 and placed on a six month inspection frequency. The DPW further de-classified the traffic loading on Louisiana Avenue, restricting it to seven tons until the repairs could be executed.

In July 2013, the City of Aurora experienced a significant storm event, which prompted the re-inspection of all culverts with an NBI of 3 or below. The post flood inspection did not reveal any roadway subsidence, but the culvert had deteriorated further with active piping of water under the culvert barrels and some failures at the culvert joints, shown in Figures 2 and 3.

Figure 2. CMP joint failure.

Figure 3.CMP invert deterioration with water piping.

This resulted in the DPW shutting down the shoulder, parking lane and sidewalk in the eastbound direction on Louisiana Avenue. The project was then elevated to emergency status.

CONDITIONS ASSESSMENT

Inspection of the culverts was done by manned entry. The extent of the voids behind the CMP were estimated by tapping the barrel with a hammer and conducting a visual inspection of the deteriorated invert. It became apparent early in the investigation that deterioration of the eastern barrel was exacerbated by the installation of the sanitary sewer line under the culvert. It seemed there wasn't adequate clearance between the sanitary sewer line and the culvert, and the sanitary line was installed using a trenchless method which compromised the invert of the culvert. The invert then corroded away causing piping and loss of pipe bedding material resulting in voids behind the culverts.

DESIGN CONSIDERATIONS

The culverts traverse Louisiana Avenue, a major City thoroughfare, and completely closing it to facilitate a "dig and replace" would not have been not acceptable to the greater community.

Wilson & Company was tasked by the City to provide a design for the rehabilitation of these culverts. The recommended rehabilitation system had to have the following minimum characteristics:

- No reduction in hydraulic capacity when compared to the existing system
- Design for a fully deteriorated design condition, thus restoring the original traffic load capacity

- Minimal impact to the environment and the travelling public
- No adverse impact to the receiving stream due to the culvert lining

Wilson & Company recommended rehabilitation of the culverts utilizing the Centrifugally Cast Concrete Pipe (CCCP) method. The loading on the liner was a summation of the following:

- Soil pressure based on the Marston theory in which the load on pipe is equal to the weight of prism of soil directly over it
- Hydrostatic pressure based on the free water surface, in this case a conservative assumption of 1-foot below the street surface was used
- Live loads based H20 truck loading

The liner was assumed to be rigid and no deflection was allowed in the design. Finally, a safety factor of two was used to determine the liner thickness. This was the basis of design provided to the prospective contractors to establish their designs.

HYDRAULIC ANALYSIS

The correct capacity of the culverts was ascertained by reviewing an existing hydrologic study on Side Creek and as-built information. HEC-RAS analyses were utilized to verify HY-8 current CMP accuracy. The discharge for the 100-year storm event at the inlet of the culvert, based on the 2009 Side Creek Drainage Study 3, was noted as approximately 1318-cfs, and all analysis for comparative CCCP liner thicknesses were based off this design discharge.

The HY-8 analysis, and verification with other methods, revealed that the culvert was under inlet control for the 100-year design flow in all proposed conditions. Due to inlet control conditions, changing the material within the culvert from a CMP to a CCCP liner would affect the hydraulic capacity of the structure.

Additional liner thickness added to the existing CMP conditions would have decreased the entrance area of the culvert and raised the headwater elevation of the channel at Louisiana. Culvert capacity analyses were completed for the existing CMP, 1-inch thick CCCP liner, 2-inch thick CCCP liner, 2.5-inch thick CCCP liner, and 3-inch thick CCCP liner. Based on hydraulic capacity analysis, the maximum thickness that could be applied, without overtopping the roadway during the 100-year storm event, was approximately 2-inches of CCCP liner. In addition, this thickness would provide the needed structural strength without adversely impacting the upstream or downstream environment.

Due to the reduction in the roughness coefficient after lining the CMP, it was determined that the velocities exiting the culvert would increase by about 30 percent. Wilson & Company determined that the current armament downstream of the culverts was sufficient, and no additional armoring protection would be required as a result of the cementicious liner.

In summary, a 2-inch CCCP liner could be added to this CMP without significant impact to the surrounding area, based on the HY-8 analysis and verified with a previous HEC-RAS analysis and FHWA nomograph investigation.

BID PROCESS

To minimize the risk to the City and create a level bidding atmosphere, the City prequalified bidders and engaged in a collaborative bid process. The City provided the basis of design and the prospective bidders then submitted their designs, with their statements of qualifications. The qualifications proposals were evaluated primarily on the proposed CCCP systems and the competence and experience of the applicators. The successful prequalified bidders and CCCP systems were:

- IPR: EcoCast System
- ACE Pipe Cleaning, Inc.: Centripipe System
- American West: Centripipe System
- Standard Cement Materials, Inc.: GeoCast System

Bids were solicited from the prequalified bidders based on 2-inch application thickness as dictated by the City. The responsive low bidder for the project was ACE Pipe Cleaning, Inc., utilizing the Centripipe System by AP/M Permaform®. Below is a summary of the bid information:

Engineer's Estimate	$ 235, 000.00
Average of Bids Received	$ 393, 725.00
Low Bidder	$ 217, 000.00

INSTALLATION OF CCCP LINER

Installation of the liner began by removing all debris from the line using pressurized water. The debris and process water was then impounded and disposed of in an appropriate manner. Once the pipes were fully cleaned, the contractor proceeded with the invert repair along the entire length of pipe, including filling the voids outside the pipe, below the spring line, with low strength flowable chemical grout, shown in Figure 4.

Figure 4. CMP invert repair.

Storm flows had to be maintained during the entire repair process, therefore only one of the culverts could be taken out of service at a time. All flows were diverted from the storm sewer being repaired into the one remaining in service until all work was completed and the line could be returned to use. Sand bags mitigated the nuisance storm water diversion so that, if a larger storm ensued, the diversion would be overcome and the nearby neighborhood would not be flooded, shown in Figure 5.

Figure 5. Stormwater control.

Grout ports were installed into the CMP to intercept any voids behind the pipe. These ports were installed per the detail in Figure 6:

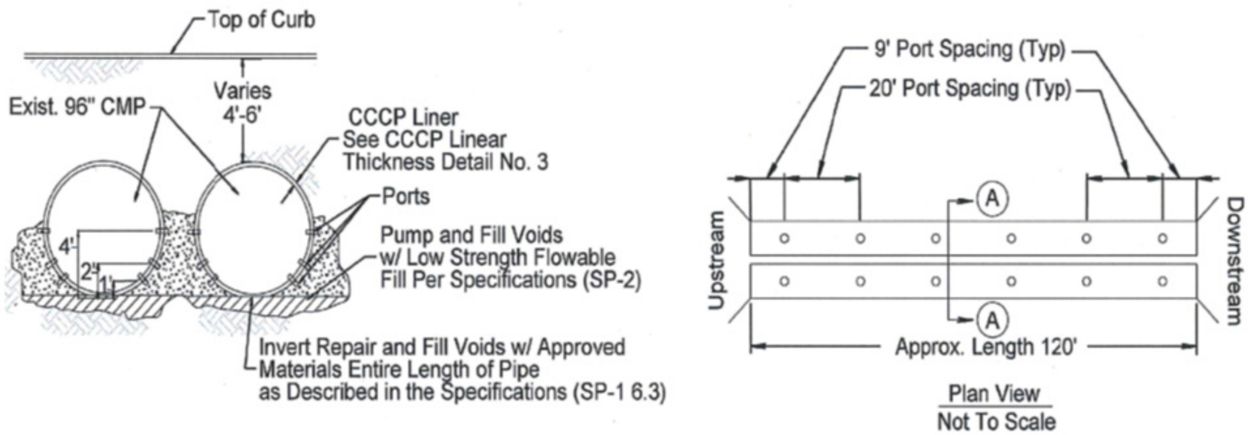

Figure 6. Grout port configuration.

Low strength flowable grout was pumped into these voids, taking care not to pressurize the grout pump above 30 psi, shown in Figures 7 and 8.

Figure 7. Grout ports.

Figure 8. Grout pumping.

The liner was sprayed on in passes of approximately ½ inch thick. Each pass was allowed to cure overnight before the next pass was applied. The finished liner and liner casting head are shown in Figures 9 and 10.

Figure 9. Finished liner.

Figure 10. Liner casting head.

QA/QC

Prior to the beginning of the liner application, 2-inch long studs were riveted into the ridges of the CMP. These served as a means to verify the specified liner thickness. Liner thickness detail is shown in Figure 11.

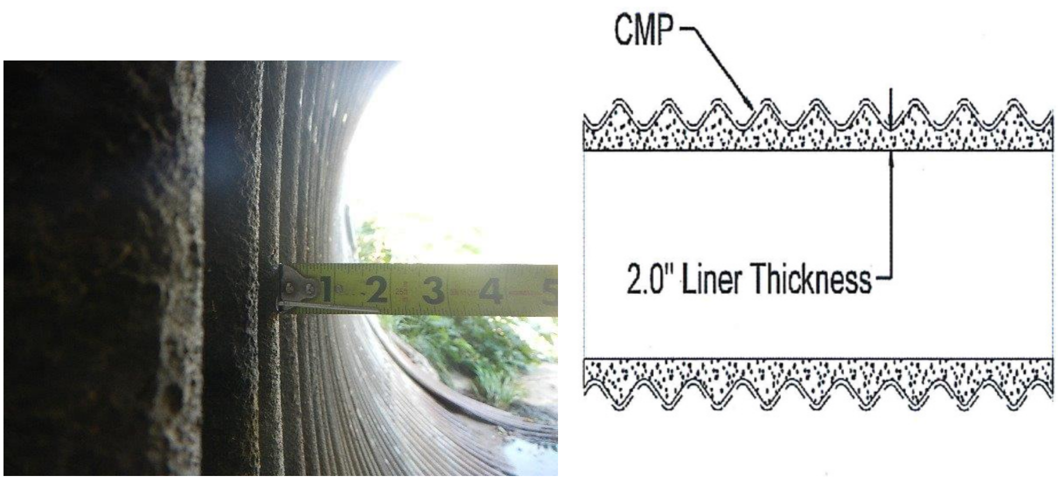

Figure 11. Liner thickness detail.

Cubes measuring 2”x2” were prepared in the field and sent to the City of Aurora materials testing laboratory to measure liner physical properties. Before shipping the material to the project site, the contractor tested one bag of material from each production batch to ensure it could attain a 3000 psi compressive strength after one day. The liner was specified to have the minimum physical properties:

- Compressive Strength (ASTM C39) or (ASTM C-109) 8,000 psi @ 28 days
- Modulus of Elasticity (ASTM C469) 3,500,000 psi @ 28 days
- Flexural Strength (ASTM C293) 800 psi @ 28days
- Bond Strength (ASTM C882) 1400 psi

Strength testing of the cubes revealed they were yielding only half of anticipated results. The City then took a bag of material and prepared and cured the cubes in laboratory conditions. These cubes yielded the anticipated results.

CONCLUSION

Centrifugally Cast Concrete Pipe is a viable and cost effective rehabilitation method for large diameter CMP. However, the industry needs to develop an ASTM so the liner design methodology can be standardized and accepted into the principals and practices of engineering. This would remove ambiguity in the design process and improve efficiencies in the design process. If cubes are going to be used for quality control, care must be taken to ensure they are prepared and cured properly, to avoid erroneous results. The CCCP rehabilitation foot print is minimal, as evident in Figure 12. Therefore, this rehabilitation method can be used in areas where traffic must be maintained. Rehabilitation utilizing the CCCP liner for large diameter CMP is a viable and cost effective option. Figures 13 and 14 show before and after images of the CCCP rehabilitation process.

Figure 12. Installation footprint.

Figure 13. Before CCCP rehabilitation.

Figure 14. After CCCP rehabilitation.

New ASTM Standards to Encourage Wider Use of Laser Profilers and Video Micrometers in Post-Construction Inspection of Pipelines

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Abstract

Laser profiling is a non-contact inspection method used to determine a pipe wall profile. A remote control tractor unit with a CCTV camera and attached laser profiling head are guided down the pipe to record defects, excessive deflection, cracks and holes. When the tractor reaches the end of the pipe run the unit is retracted in the rotating laser system, while projecting the laser onto the interior of the pipe to complete optical triangulation. In a non-rotational projection system, the 360 degree laser ring is video recorded as the camera and the laser projector are retracted. A real time report including video, three dimensional wireframe and deflection analysis are immediately available for review by contractors, engineers, owners, and state inspectors. Given that the laser profiling can deliver the deformed shape of the pipe cross section, at a pitch of as small as 0.2 inch, the use of Spangler's equation and the E' is likely to fall out of favor. Two ASTM standards F3080-14 and F3095-14 have been written while the third is in progress within ASTM F36. The authors are encouraging wider use of this novel technology within the pipeline industry.

INTRODUCTION

Laser profiling provides an effective method of mapping defects in municipal-stormwater, potable water and sewage, highway drainage, and other pipeline systems. A laser profiling system containing a transporter, a closed circuit television camera (CCTV), and either a non-rotational laser projection system or rotating laser diodes, travels along the interior of a pipe and records deformed shape of the pipe cross section, all joint gaps, cracks and observable defects. The pipe need not be dry or free of debris for the technology to work due to evolving capability of the software to mask such anomalies. By knowing the position of the rotating laser with respect to the camera at a known focal length or distance and establishing which part of the sensor the light spot is viewed with the use of software, the location is translated into a pixel point, one can find the exact location of the interior pipe wall with respect to the laser and camera. The radial distance data along the pipe is plotted yielding a three dimensional wire frame image of the pipe. Using pan and tilt features, any anomalies, such as crack width, can be viewed and analyzed.

Lack of consensus standards and attacks on the accuracy and repeatability of measurements have been the primary content of all campaigns against the use of such modern inspection methods. The authors set out to develop three companion ASTM standards within the technical committee F36 and succeeded publishing two out of three in 2014.

WHAT IS IN ASTM 3080-14, ASTM 3095-14 AND WK 45911?

ASTM F3080-14: This practice covers the procedure for the measurement to determine any deviation of the internal surface of installed pipe compared to the design. The measurements may be used to verify that the installation has met design requirements for acceptance or to collect data that will facilitate an assessment of the condition of pipe or conduit due to structural deviations or deterioration. This practice applies to all types of pipe material and construction or shape and to storm sewers, drains, sanitary sewers, and combined sewers with diameters from 6 to 72 inches.

ASTM F3095-14: This practice covers the procedure for the post installation verification and acceptance of buried pipe deformation using a visible rotating laser light diode(s), a pipeline and conduit inspection analog or digital CCTV camera system and image processing software. This practice applies to all types of material, all types of construction, or shape and to storm sewers, drains, sanitary sewers, and combined sewers with diameters from 6 to 72 inches.

The third standard within work item 45911: This standard practice will cover the minimum requirements on means and methods for the application of Video Micrometer for the measurement of cracks, joint gaps and other measureable visual abnormalities that could affect performance or life of pipes, conduits and culverts.

PUBLIC AGENCIES SEEK BETTER INSPECTION TOOLS

The movement to embrace laser profiling and video micrometer measurements is driven by state departments of transportation (DOT) s, American Association of State Highway and Transportation Officials (AASHTO) and Municipal Utilities. The users of laser profilers and video micrometers are far more knowledgeable and sophisticated than portrayed by the pipe industry. When Peev et al. (2011) were retained by Michigan DOT for the comparative evaluation of available equipment on the market, the following selection criteria were used:

- Ability to measure pipe diameter deviations, pipe cracks and other pipe anomalies
- Quality and expediency of results/reports, quality of analog CCTV
- Precision and necessary calibrations
- Laser device technology, field operation procedures and safety issues
- Resolving laser profiling issues related to presence of pipe corrugations
- Pipe size ranges and approximate no-laser data zones at end of pipe runs
- Measuring slope with inclinometer and option to record speed when laser profiling
- Equipment pricing and possibility for upgrade to digital camera

In addition, the following minimum equipment capabilities from the MDOT Special Provision *" Laser Inspection of Sewer and Culvert Pipe"* were considered:

- Optical zoom (min 10:1) and combined digital/optical zoom (min 40:1)
- Adjustable transporter speed and adjustable camera height
- Distance counter; inclinometer
- CCTV Camera capable to rotate 360 degrees and to pan and tilt 90 degrees
- Camera ability to pan and zoom 360 degrees at every joint and pipe anomaly
- Minimal size of cracks to be measured (0.01")
- Profiler accuracy (0.5%); profiler repeatability (0.12%)

Another example of the admirable capability of state DOT engineers can be inferred from the strict guidelines that are established in the current version of the FDOT specifications (2013) for the final inspection of pipe culvert systems. Among the criteria that must be examined are pipeline grade, the proper sealing of all joints, minimal pipe deflection, and freedom from cracks, and other observable defects - all with the noble objective of the owners wanting to "do more with less" by expecting the pipe industry to furnish better quality pipe and the installers to avoid poor workmanship during construction needing hundreds of millions of dollars of new pipe requiring expensive pipe repair.

HISTORICAL BACKGROUND OF LASER PROFILING

Laser profiling was originally developed as a method to inspect the placement of cured- in-place pipe systems (Hancor, 2007). With the adoption of laser technology in culvert inspections more efficient means of locating and identifying defects in reinforced concrete pipe was born. Holdener (2011) questioned, however, the accuracy and repeatability of the results from laser profiling and video micrometers. Although analysis of the severity of a defect and the determination of a proper course of action remain the responsibility of the engineer of record, the accurate and precise detection of these defects for municipal storm water pipelines is crucial in preventing costly and unnecessary repairs (Bennett and Logan 2005). FDOT outlines inspection criteria for newly installed municipal storm water pipe per Section 430 of their Standard Specifications for Road and Bridge Construction. FDOT is entitled to "For pipe 48 inches or less in diameter, provide the Engineer a video DVD and report using low barrel distortion video equipment with laser profile technology, non-contact video micrometer and associated software that produces:

1. Actual recorded length and width measurements of all cracks within the pipe;
2. Actual recorded separation measurement of all pipe joints;
3. Pipe ovality report;
4. Deflection measurements and graphical diameter analysis along x and y axes;
5. Flat analysis report;
6. Representative diameter of the pipe;
7. Pipe deformation measurements, leaks, debris, or other damage or defects;
8. Deviation in pipe line and grade, joint gaps and joint misalignment.

Often the concrete pipe industry has used autogenous healing of cracks as a deterrent to owners, engineers, or contractors scrutinizing cracks. Edvardsen (1999) states, “The most significant factor which influences the autogenous healing is the precipitation of calcium carbonate.” The two items that most influence autogenous healing process are the width of the crack and the prevailing pressure of the water. Edvardsen finds that the largest permissible crack width for autogenous healing under the lowest prevailing hydraulic influence is 0.25mm or ~0.01”. This is why the use of quality video micrometer is so important. The proposed ASTM standard within work item 45911 needs to be in-line with this permissible crack size. There is some concern that concrete pipe in storm sewer use may not have the necessary hydraulics to form the calcium carbonate, particularly if the crack is not in the flow line and had limited to no sustained contact with water.

It is worth mentioning two significant milestones in laser profiling. Australian authorities developed the Pipe Inspection Real time Assessment Technique (PIRAT, 1995) with laser and sonar scanners, and contained “two semi-independent systems” that collected and interpreted data. This system also included processing software. The Sewer Scanner and Evaluation Technology (SSET) system that was developed in Japan incorporated the video recording function with a gyroscope and an optical scanner. The data processing involved the use of image filters but the added implementation of the gyroscope, in conjunction with the optical scanner for data geometry recognition, helped account for the problems driven by the mobility.

LASER PROFILING IN USE

In 2005, an investigation of existing drainage systems throughout the states of Kentucky and Ohio took place to evaluate the performance of high-density polyethylene (HDPE) pipes. These inspections were performed with CCTV cameras and laser profiling equipment – and although crack detection was done primarily with video recordings, the laser profilers provided valuable information in identifying pipe distortion, including vertical and horizontal deflections. This effort suggested laser profiling into pipeline inspection. In addition to Florida, other state DOTs that have either started requiring laser profiling or are investigating the technology for use in future installations include California, Kansas, Kentucky, Michigan, Minnesota, Missouri, North Carolina, Ohio, Pennsylvania, Texas, Utah, and Virginia.

While the momentum grows for the use of laser profiling, there is a parallel movement presenting resistance to change for the better from the pipe industry demanding those advocates of laser profiling and video micrometer measurement must provide standardization of this new technology (Holdener, 2011). Although the value of standardization was emphasized, peers from the concrete pipe industry, however, have been obstructive to the efforts of the members of ASTM F36 from completing a standard that all users from this committee are eager to see in print for wide spread use. The authors of this paper are of the opinion that there is no consistency or a standard set forth by an industry organization on accuracy of crack width measurement, and it is false to assume that cracks of less than 0.05 inch (1.3 mm) are insignificant."

AASHTO AND STATE DOT REQUIREMENTS

Discussions concerning the use of deflectometers and mandrels in detecting and testing pipe deflections began in June 2005, among the engineers within FDOT, but by September of the same year, they were considering the potential benefits from the implementation of a "laser ring" inspection method. By this time, AASHTO had become familiar with laser profiling. In 2007, FDOT Specifications included the requirement of laser scanning for final inspection of all newly installed pipe culverts. As part of FDOT's requirements on laser profiling equipment, an accuracy of +/- 0.5% was expected with the readings. Larry Ritchie (2014), a leader in pipe inspection from FDOT, stresses in his lectures to large audiences "*Over the last 5 years, the Department has spent approximately 175 million dollars on drainage pipe and over 9.5 million dollars on pipe repair. With costs like these, it is extremely important to ensure that pipe is installed correctly, inspected thoroughly and replaced or repaired correctly when warranted.*" Minchin (2014) demonstrates, for example, FDOT's proactive efforts.

HOW DO LASER PROFILERS WORK?

A laser is made up of three main elements: an active material, an energy source, and a pair of mirrors. The two mirrors, one completely reflective and the other semi-reflective are used to further propagate electron excitation and to direct laser light through the aperture. An image laser profiling system traversing along the interior of a pipeline is depicted in Figure 1. A modern laser profiler can identify attributes such as pipe ovality, and horizontal and vertical deflection. Laser profiling comes in two forms: either laser ring or rotating laser. The first method entails projecting a laser ring along the interior of the pipe culvert, just ahead of the CCTV camera. As the unit travels along the pipe invert, any defects in the overall shape of the pipe are recorded both visually and digitally. The laser ring method performs 1080 measurements of the pipe radius per image while 30 images are captured and stored every second. The reports contain excessive deflection, thinning of the wall due to sulfide corrosion, debris level, capacity calculations, post lining quality control, internal spray coating thickness gauging, crack widths and lengths, sizes of joint gaps and other. A laser ring projected onto the interior wall of a pipe is shown in Figure 2. At the specified speed of longitudinal travel of 30 ft/min, the profiling software in the non-rotational system can process nearly 2 million measurements per minute. Peev et al. (2011) wrote in their work for Michigan DOT "*When used at 30 ft/min, the spinning laser measures in a 4.8" inch long spiral. Even this measurement frequency produces enough pipe diameter data for practical pipe evaluation*." The rotating head profiler has two laser diodes built into the CCTV camera head as shown in Figure 3. The diodes take continuous measurements while the camera head rotates 360^{o} at a set speed while the crawler is pulled back through the pipe as shown in Figure 4.

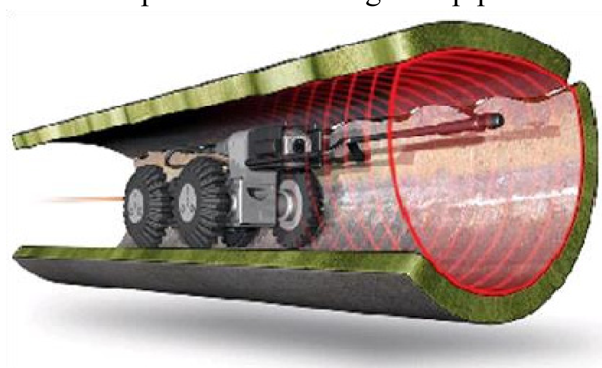

Figure 1. Laser profiler along pipe *(from AET Robotics and Inspection Services)*

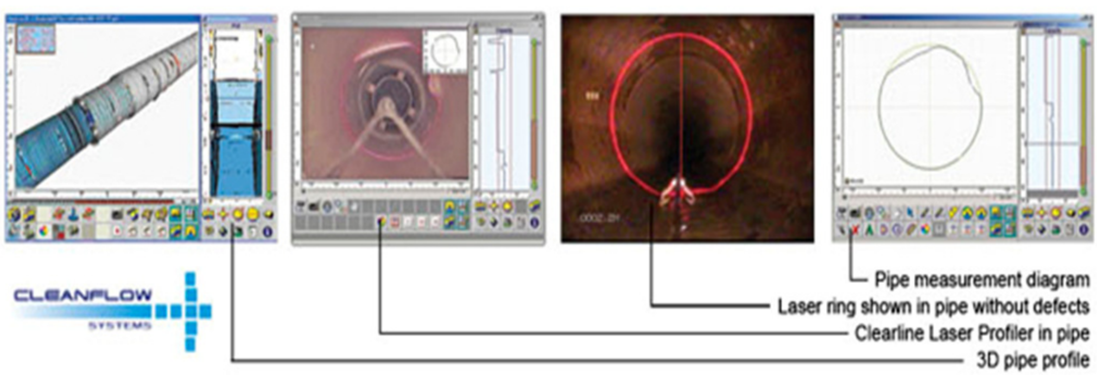

Figure 2. Laser ring profiler *(from CUES, Inc.)*

While the unit travels down the length of the pipe, the lasers are aimed towards the pipe surface, remaining orthogonal to the wall, and the laser mount rotates at a predetermined speed producing a spiral image recording pipe deflections, diameters, and deformations. Since the laser diodes are integrated into the camera unit, no user calibration is necessary prior to pipe. This efficient system is also called "rotational laser diodes" as shown in Figure 4. The equipment is factory-calibrated for the rotating laser diode type and will immediately measure, record and correctly adjust to the pipe diameter. The calibration is done, however, before each run for non-rotating laser projection technology and a typical scan is shown in Figure 5.

Figure 3.Two laser diodes embedded in CCTV camera head *(from FDOT)*

Figure 4.CCTV – Rotational laser diodes camera *(from Rausch USA)*

Figure 5.Laser scan of a brick sewer *(from CUES, Inc.)*

ACCURACY AND REPEATABILITY

The practical accuracy of most laser profiling systems was questioned by Holdener (2011) when he wrote "*the majority of manufacturer literature boasts an accuracy of 0.03 inch under ideal laboratory conditions. Comparing that with the 0.01 inch requirement in the field can put into question the validity of most profiling systems currently used in Florida.*" It is significant enough to point out that the proposed standard on the proper use of video micrometer for crack and joint gap measurement received multiple negative votes demanding that the requirement on accuracy be relaxed from the meaningful 0.01 inch to an arbitrary 0.05 inch. The ASTM F36 Task Group members responsible for wk 45911 choosing 0.01 inch for accuracy is well supported by the following: As required per Section 449 of the FDOT Specifications, ASTM C76 – 11, and AASHTO Section 27, cracks identified as being 0.01 inch in width and at least 12 inches in length would not be accepted. FDOT does have specific requirements mandating the calibration of all laser profiling equipment. Resolution can be defined as the minimal distance that can lie between two points and still have those points register as distinct, individual points (Cullity and Stock, 2001). The ultimate resolution is a function of the laser's wavelength - hundreds of nanometers (~$4x10^{-6}$ inch).

The type of pipe to be examined should also be taken into account as different materials can absorb, reflect, or refract the probe laser in different ways, altering the perceived resolution (Hummel, 2001). Even the roughness of the surface of the pipes can have an effect on laser reflection. The resolution will determine the minimal size crack that can be detected at a given distance under nominal conditions (Chu and Butler, 1998). This information should be available from the manufacturers' literature in the form of a "Distance versus Resolution" graph, or even as an equation. Information presented about the assumed conditions and angle of detection should be provided, as this information will help determine how applicable the given data is to the project. Knowing the angle of detection may require more investigation depending of the diameter of the pipe being examined and the location of the probe source inside the cross section of the pipe (i.e., centered versus non-centered).Even given all the required information from the manufacturer, independent testing and confirmation of the resolutions for a wide set of conditions should be performed and the data obtained used to gauge the accuracy of the manufacturers' claims.

EQUIPMENT LIMITATIONS

Improper configuration and poor initial positioning of the equipment can result in inaccurate data (Dettmer et al., 2005). Buonadonna et al. (2011) summarized the most recurring problems with laser profiling as follows: a) the laser will only collect information above a waterline (as the laser light will be refracted); b) the laser cannot distinguish between material densities; c) it is difficult to align the laser "cross-section" with the pipe center. Faulty data can result in distorted images that appear "cloudy" or do not have corresponding data points.

OPERATOR LIMITATIONS

Laser profiler manufacturers, concrete pipe suppliers and those in the video inspection industry are quick to point out the need to improve the operator's knowledge and implementation of existing specification guidelines. For example, many have observed the disregard of the maximum system speed (30 feet/minute) for running a laser profiling inspection. In some cases, the laser profiling systems being used do not display the unit's longitudinal travel speed on the video screen, and the operators are quick to exploit this feature. When an inspection is performed too fast, the resulting images on the CCTV video recording will appear blurred and will not help if an image must be referenced with a corresponding laser profile. Several CCTV pipe inspection systems currently offer an optional speed display within the recorded video image. Other instances of inspector failure include the omission of joint gap reports for all connections along a pipeline run. Particular operators will only supply gap reports for those instances in which the requirements are not met. Although a subsequent joint gap may fall within the appropriate parameters, remediation for an adjoining joint gap may affect the connection of the previous joint gap. Therefore, it is vital to have all the information available.

There is a direct correlation between inspection run speed and the data analysis capacity for any given laser profiling equipment. CUES Laser Profiler System states that their pipe ovality routine processes at a maximum speed of 30 times/sec. When considering the average pipe culvert segment length of 8 feet and assuming the maximum inspection speed of 30 ft/min, completing a run for a single segment of pipe will take approximately 16 seconds resulting in 60 analyses/ft. Processing speeds and individual operator inspection speeds must be verified and a standardized minimum number of analyses should be established. The new ASTM standards F3080-14 and F3095-14, by the F36 technical committee put to rest these questions and many more to pave the way forward for wide spread use of laser profiling.

VIDEO MICROMETER TECHNOLOGY

Laser profiler manufacturers use video micrometers to measure crack and gap sizes. A video micrometer, typically attached or built into a laser profiling setup, consists of two parallel lasers. These lasers are spaced apart at a known distance, and they function as a reference point when measuring the required defect. Proper measurement of a pipe defect necessitates an accurate alignment of the CCTV mechanism. Using the image as a guide, an operator must maneuver the recording device so that the image and respective laser beams are perpendicular to the longitudinal axis of the pipe. When an image of the defect and the two laser points are properly captured on the screen, a ratio can be established between the screen image pixilation and the known reference distance between the two lasers.

Digital and optical zooming provides a magnification of as much as 120:1 for the operator to readily discern cracks as small as 0.01 inch wide or smaller. Arguments built on counting pixels by asserting that the human error makes it impossible to measure crack widths to 0.01 inch accuracy is as flawed as claiming microscopes or telescopes do not work. It is imperative that the two lasers form a 90 degree angle with the pipe to ensure there is no skewing of the laser lines resulting in erroneous defect measurements. Crack detection involves a record of the precise location of the defect along the pipe's longitudinal axis and along its circumference, and must detail the width and the length, if exceeding the dimensions for a minimum-sized crack.

NEW PIPELINE DESIGN METHOD USING LASER PROFILING

The new pipeline design method entails: a) a test section of the pipeline at the site in representative soil conditions, b) performing laser profiling of the response of the test section along its length c) designing the pipeline using the data from the test section d) verifying the behavior of the actual pipeline during and after construction and e) implementing the lessons learnt on future pipelines. This migration from the office out into the real world to design, construct and inspect pipelines once the pipe fabricator and the contractor are ready for project acceptance by the design engineer would be a much needed shift in paradigm in the pipeline industry. Laser profiling in post construction inspection can ensure that the contractor indeed has used good quality in workmanship and has met the engineer's specifications.

CONCLUSIONS

Laser profilers and video micrometers offer major improvements in our ability to design, construct and monitor pipelines, conduits, culverts, sewers and highway drainage systems. The ASTM standards the authors have developed with the body of knowledge of the 150 members of ASTM F36 technical committee bring instant credibility to laser profilers and video micrometers to pave the way for their broader use. These ASTM standards also reduce the amount of time it takes engineers to write bidding documents and technical specifications. These standards bring an added degree of comfort for the engineers, contractors and the users knowing that the thorough vetting process built as part of the consensus building within ASTM is based on the balanced representation of consumers, users, producers and those of general interest. Standards ASTM F3080-14 and F3095-14 help to form contracts between buyers and sellers. In case of disagreements or disputes, standards form the backbone of establishing the "standard of care" in our judicial system. In a way, the buyers and sellers have the standards provide a preview of what case law is likely to be written and help them become aware of how to avoid errors and omissions. Given that the plastic and corrugated metal pipe industry did not interfere, these two ASTM standards reinforce the notion that these two segments of the industry continue being early adapters of new technology. The drivers that fuel either growth or decline of one segment of the pipe industry against another are: nimble enough to recognize trends and react swiftly; adapting to changing customer demands; superior vision and the ability to execute the mission flawlessly, compared to competing players; a mindset toward quality improvement in the products offered to the end buyers. The most efficient and transparent manner in which producers meet the goal of "quality improvement," is to encourage the development and dissemination of ASTM standards covering the use of new technologies for quality assurance to their buyers.

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Understanding the Benefits of Multi-Sensor Inspection

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Abstract

Over the past several years, multi-sensor technology has provided the tools to finally overcome many of the obstacles that were previously encountered inspecting large diameter interceptors. There are countless combinations of collection system materials, sizes, shapes, and conditions that require innovative inspection technologies. Continually, more engineers and municipalities are utilizing these technologies to make more informed, cost-effective rehabilitation decisions. Inspections can now be economically performed using high definition (HD) cameras and a variety of sensors for above-water and below-water data collection. While inspection costs are a fraction of rehabilitation costs, the condition assessment provides invaluable information in the design process. However, different rehabilitation methods require different types of data to make informed design decisions. When all of these data sensors are contained on a single delivery system, the inspections can be economically performed in a single inspection run, but what sensors do I need? What is the difference between standard CCTV and HD CCTV? Do I need 2D or 3D laser data? What are the different types of sonar technologies? Do I need to measure gases and temperature in the line? What does an inclinometer tell me? There is no "one size fits all" when it comes to multi-sensor inspection technologies, so it's important to understand the basic mechanics of different sensors and how the collected information will benefit the design engineer. With all the new innovative technologies in the market today, it's important to understand the benefits, not necessarily the features. Using actual case studies, the presentation will provide information on the operation of the different data collectors and delivery systems by different manufacturers. Identifying conditions prior to the inspections to enable planning of the most effective means of performing the work and those conditions that may be encountered that could cause problems are discussed and illustrated. Sample reports and typical illustrations will also be presented.

1. INTRODUCTION

Throughout the United States, concerns abound regarding our infrastructure, especially our buried infrastructure that has been in operation for decades. Many of those concerns focus on reinvestment and rehabilitation of those facilities that are known to be deficient or near the end of their useful life. However, there's a considerable portion of the infrastructure where the assessed condition is unknown largely due to lack of reliable inspection technologies. This has been particularly true in the wastewater industry, particularly large diameter sewers. Although millions of feet of sewer lines are televised in the United States each year, larger interceptors that carry significant flow are often ignored because of accessibility issues, lack of redundancy, safety concerns, illumination, cost, clarity of information and the difficulty and cost of dewatering.

Since 2008, multi-sensor technology has provided the tools to finally overcome many of the obstacles that were encountered in the past. More and more communities are using this type of technology to inspect sewers and make more appropriate rehabilitation decisions, which continually proves the cost effectiveness of this value-added tool. To date, millions of feet of large diameter sewers have been inspected using multi-sensor technology.

Using a combination of a high definition camera and various combinations of laser technologies above the water surface, observations of corrosion, deflection, ovality, missing brick courses, damaged pipes, poor bedding, etc. are recorded. Below the water surface, sonar technology identifies the depth and volume of debris and major structural anomalies without the need for expensive dewatering systems. When all of these data collectors are contained on a single delivery system, the inspections can be economically performed in a single inspection run.

The collected data is processed into a single submittal with videos simultaneously presenting the laser above the water surface and the sonar below the water surface. The camera data is submitted in a video per National Association of Sewer Service Companies (NASSCO) Pipeline Assessment and Certification Program (PACP) protocol that can be incorporated into most municipal databases. Data reports with still photographs, computer-generated drawings and findings at their specific locations supplement the videos.

2. BACKGROUND

The management and assessment of our wastewater infrastructure is a critical component in the preservation of the environment, economy and overall public health. Drinking water and wastewater infrastructures are the lifeblood of our society and run throughout our communities, near our homes, schools and businesses. Due to the age of our infrastructure, many utilities need to conduct extensive condition assessment and rehabilitation; however, many utilities lack the experience and capabilities to perform such work. In addition, local budgets are often insufficient to fully address these comprehensive assessment needs. According to the Water Environment Research Foundation (WERF), it appears that, when selecting a technology for condition assessment, most utilities are primarily focused on cost due to their restricted budgets. Although large diameter interceptors and trunk sanitary sewers are often most critical, they have received relatively little attention in terms of operation review and assessment (Sinha, et al. 2013).

Over the past few years, an initiative funded by WERF and the Environmental Protection Agency (EPA) has resulted in the development of a national, interactive database of assessment technologies. This database is called the Water Infrastructure Database (WATERiD). Based on information gathered for this database, several consistent themes were identified from utilities across the country:

- For the most cost-effective renewal work, proper condition assessment is very important.
- Condition assessment is a part of asset management, and effective asset management is more critical than simply locating leaks.
- Collection and review of past assessments are important, as previous condition assessment data provide a baseline for future assessments. Therefore, data management is critical.
- Using a combination of condition assessment technologies results in a more complete and useful picture of a pipeline's condition than using a single condition assessment technology.
- At times, outsourcing condition assessment work is better than performing the work in-house, in terms of both performance and cost.

The scope of WATERiD is to provide a platform whereby institutional knowledge can be shared for all inspection technologies across all water and wastewater systems. However, these identified themes are also specifically applicable to large diameter gravity interceptors and trunk sewers. Failure of these "main arteries" can be catastrophic and extremely expensive. Rehabilitation of large diameter sewers can cost as much as $1,500 per linear foot, or more after a collapse or major failure. Based on these figures, it is prudent to spend pennies on the dollar to inspect interceptors for accurate condition assessment and rehabilitation design data.

3. DISCUSSION

The EPA defines condition assessment as follows (Feeney, et al. 2009):

The collection of data and information through direct inspection, observation and investigation, indirect monitoring and reporting and the analysis of the data and information to make a determination of the structural, operational and performance status of capital infrastructure assets.

The primary purpose of condition assessment is to detect pipe defects that may indicate the likelihood of failure, as well as assess the collection system's performance. Based on this premise, condition assessment hinges on data. Before you can decide on investigative or inspection techniques, you need to decide what type of data you need and how will that lead to an accurate assessment of condition. In terms of interceptors and trunk sewers, the need to ascertain condition is paramount:

- Failure of these "main arteries" can be catastrophic
- Identify sources of inflow and infiltration (I/I)
- Prioritize preventative maintenance activities
- Determine connectivity of the collection system
- Identify structure defects
- Rehabilitation is expensive, especially if reactive

Currently, the available inspection technologies include visual, acoustic, laser, temperature, gas sensors, electromagnetic and others. In isolation, they can all offer good data, but limited in terms of overall assessment. The combination of any such technologies offers the most comprehensive, complete assessment. Before one can appreciate the overall value of multi-sensor inspection, it is important to understand the each technology/ sensor and limitation thereof.

Visual Inspection – Still cameras or video cameras have been the most tried and true inspection technology for gravity sewers for decades. Hundreds of millions of feet of pipe have been inspected with this technology. It is a great tool to see what the human eye cannot. It provides visual data on leaks, location of service laterals and observations of structural defects and sediment levels. Whether it is accomplished by physical manned entry or robotic cameras, visual assessment is a tool that will always be coveted and add value. However, it is largely a qualitative assessment tool that is limited to identifying only visible defects. Safety, accessibility, lighting and camera clarity are often concerns and limitations for large diameter sewers.

Acoustic Inspection – Acoustic inspection technologies have been around for decades, yet still continue to offer some of the most innovative applications. Basically, this subset of technologies is focused on sonar, ultrasonic, echo and sounding technologies. Sonar is the most widely recognized tool because it is used across multiple industries (beyond water/wastewater) for below-water data collection. Sonar technology is based on time measurement of sounds bursts and is largely used in fully submerged pipes or siphons to measure debris levels and lost capacity. The real benefit of sonar is that you can inspect pipes in their in situ condition without bypass pumping or dewatering. Sonar is a great tool, but it is limited in terms of precise identification and location of defects, particularly longitudinal. Also, sonar sensors can only provide data below the flow line.

Laser Inspection – Laser-based technologies are precise, yet practical tools that are often misunderstood. In simplistic terms, lasers measure light, either in terms of distance or frequency. Lasers can determine the shape of the pipe, measure ovality, vertical deflection and wall loss due to corrosion. There are several different types of lasers, but generally categorized as either 2D or 3D lasers, which has to do with how many planes of data are captured by the laser.

Ring laser is a 2D technology used largely in laser profiling applications by projecting a continuous line of light onto the internal circumference of a pipeline. This is the most common, widely used laser technology for sewer condition assessment. The laser ring is recorded by a camera, either a traditional CCTV camera or a dedicated camera as part of the system. There are many different ring laser manufacturers and the majority of off-the-shelf lasers are designed to profile pipes less than 72 inches in diameter and are highly dependent on pipe material reflectivity. Two potential caveats that should be noted when using ring laser data:

i. Since ring lasers are a 2D technology, there is no "Z" component to reference each laser ring/ slice to consecutive rings. This limitation means that ring laser data cannot be used to determine bend radius or vertical offsets. It also means that if the laser is not positioned perfectly within the pipe, then the camera will record a skewed laser ring, which is commonly referred to as "skew effect." This is mostly overcome by evaluating the voluminous amount of data captured and throwing out the skewed data. There are also steps that can be taken in the field to reduce the amount of collected skewed data.

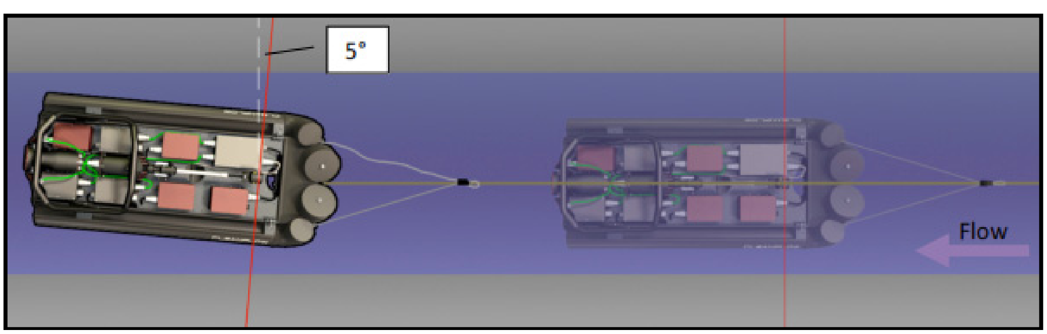

Figure 1. Illustration of Skew Effect in 2D/ Ring Laser Data Collection

ii. The accuracy of ring laser data is largely dependent on how the data is captured in the field and how the data is processed. Resolution of the camera recording the laser ring directly affects accuracy. Different CCTV camera manufacturers have different barrel distortions, so it's important to understand the equipment used when processing the data. In many cases, the CCTV operator or data processor chooses the most distinct light ring with the most constant color and intensity.

Broadly speaking, the other category of lasers are 3D lasers. The most common 3D laser used in wastewater condition assessment is a Light Detection and Ranging (LiDAR) laser. LiDAR refers to the method in which the data is captured by time measurement of light photons, not specifically being 2D or 3D. LiDAR data is precisely accurate, more so than ring laser data, but it is also highly dependent on quality data collection in the field. Depending on the inspection platform, LiDAR data can either be captured continuously and via a stop-and-scan method. With the significant amount of data captured with this technology, there are nearly countless ways to present the data and data processing is often very time intensive. An accurate 3D model developed from LiDAR data is called a point cloud and may be a good starting point for data presentation.

Figure 2. Colored point cloud model developed from 3D LiDAR laser data

Multi-Sensor Inspection – Any combination of advanced sensors, such as CCTV, sonar or laser technologies is considered a multi-sensor inspection (MSI). The real value of MSI is that data is captured both above and below the flow line and inspections are performed in situ. This offers the most comprehensive inspection data. Some of the benefits of MSI are:

- Image Quality – Most Traditional CCTV cameras are designed for smaller diameter pipe inspection. For large diameter interceptors or trunk sewers, it is important to have sufficient lighting and camera resolution to capture all visible defects and details.

- Quantification of corrosion – Since laser data is more quantitative than CCTV data, it can be used to determine pipe wall loss due to corrosion. The laser data is effectively compared against as-built drawings or laser sections of know good quality. This data is used to determine location and severity of the problem.

- Quantification and location of debris/ sediment – Cleaning large diameter trunk lines is expensive and should never be undertaken without estimating how much debris is in the line. Depending on location within the country, many interceptors have been installed at relatively flat grades, which can lead to accumulation of sediment. A sonar inspection is an inexpensive method to quantify debris and ensure cleaning budgets are spent efficiently.

- Long, continuous inspections – Many large diameter trunk lines, especially on the east coast, were installed before the development of modern standards for manhole spacing. It's not uncommon to find access points several thousand feet apart. Many MSI platforms have long-range capabilities, some up to 10,000LF in a single setup.

4. DATA PRESENTATION

Prior to deploying an MSI assessment, it is important for the utility, engineer and contractor to all be on the same page in terms of why the assessment is being performed. All MSI data can be used for condition assessment, but depending on the technologies used and how the data is collected, MSI may also be used for design purposes. Specifically, 3D LiDAR data can be used to generate Computer-aided Design (CAD) models if the data is captured properly. It is also important to distinguish between features and benefits. Most individual MSI sensors can be purchased off the shelf and the collected data should be open source. Rehabilitation decisions are based on good assessment data, not the inspection platforms that collected that data.

Depending the technology deployed, MSI data can be as much as one megabyte per foot of inspection, so presentation of the data in a simplified, meaningful way is important. NASSCO offers a good protocol for standardization of CCTV data, but there is no such industry standard for MSI data. Therefore, it is incumbent on the utility and engineer to have a cursory understanding of the type of data collected and how it can be most efficiently presented.

5. CONCLUSIONS

Throughout the United States, there are countless combinations of interceptor and trunk sewer collection system materials, sizes, shapes, and conditions that require innovative inspection technologies. Although millions of feet of sewer lines are televised each year, larger interceptors that carry significant flow are often ignored for a variety of reasons. Access, safety, illumination, cost, lack of redundancy, clarity of information and the difficulty and cost of dewatering are some of the reasons that prevent inspection of these important collection system components. Not only are inspection technologies becoming more available, but also the combined processing of multi-sensor data allows for more accurate condition assessments that were not even practical a few short years ago. The real power of the collection of multi-sensor data is the ability to process the data into a single, comprehensive, easily interpreted submittal.

Cities and municipalities understand that one of their most valuable assets is their sewer system. Today more than ever, communities realize the importance of knowing not only where their buried sewer infrastructure is but also the condition of these assets. They also understand that it is more cost effective to be proactive with rehabilitation or replacement of a sewer that is structurally unsound rather than after a catastrophic failure has occurred.

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Looking Past the Pipe Wall: Quantifying Pipe Corrosion and Deterioration with Pipe Penetrating Radar

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Abstract

Pipe Penetrating Radar (PPR) is the underground in-pipe application of GPR, a non-destructive testing method that can detect defects and cavities within and outside mainline diameter (>18 in / 450mm) non-metallic (reinforced concrete, vitrified clay, PVC, HDPE, etc.) pipes. The key advantage of PPR is the unique ability to map pipe wall thickness and deterioration including voids outside the pipe, enabling accurate predictability of needed rehabilitation or the timing of replacement. This paper presents recent advancement of PPR inspection technology together with selected case studies. Two case studies are discussed in detail. The Del Norte Trunk Sewer in Stockton, CA is a 36" reinforced concrete pipe with a 0.7" thick fiberglass liner. The objective of the PPR survey was to determine the condition of the approximately 55 years old lined RC pipe by mapping its wall thickness, rebar cover and detecting voids and/or other anomalies within or outside the pipe wall. The pipe experienced failures in the past and the fiberglass liner has at places also separated from the original RC pipe wall. PPR results confirmed 3.9 to 4.5 inch remaining wall thickness including the grouted fibreglass liner with little variation over the inspected length. Rebar cover appeared to be sufficient with no void type anomalies on any of the inspected lines. A 120 inch diameter brick lined combined sewer pipe was inspected with PPR. The pipe was built in 1906 and experienced wet weather overflows. In order to design the most appropriate rehabilitation strategy the knowledge of voids outside the sewer was critical. Over 6,000 ft of high resolution line data were collected via manned entry. PPR data revealed voids both outside and within the pipe wall and thus provided engineers the information needed to take the appropriate approach to rehabilitate the pipe. With limited available funding and budget constraints becoming more prevalent, timing of rehabilitation and overall intelligent asset management is more critical than ever. PPR provides engineers and utility owners the information to accurately estimate the remaining life left in a pipeline, refine timing of repairs, and ultimately better allocate funding for asset management.

INTRODUCTION

Pipe penetrating radar (PPR), the in-pipe application of ground penetrating radar (GPR) is one of the most promising quantitative pipe condition assessment technologies to emerge in recent years. With most of the underground pipe infrastructure reaching the end of their design life there is a need to provide measurable data in order to establish the extent of rehabilitation required or the timing of replacement for large diameter critical pipe lines.

Although Closed Circuit Television (CCTV) inspection methods are effective and widely available tools for identifying visible defects on the internal wall of pipes, CCTV cannot see behind the pipe's inner surface, nor can it quantitatively determine the extent of corrosion. PPR technology allows the implementation of proactive preventative maintenance procedures for non-ferrous wastewater and water underground infrastructure. The combined application of PPR, CCTV and LIDAR provides the most complete and state of the art inspection technology to enable proactive asset management and allow utility owners to plan and schedule the inspection and rehabilitation of critical utilities prior to the occurrence of emergency scenarios.

This paper highlights the benefits of using PPR. Examples to illustrate the key benefits are drawn from two projects, one conducted with a robotic platform the other via manned entry.

OVERVIEW OF PPR IMAGING TECHNIQUE

Ground penetrating radar is the general term applied to techniques that employ radio waves to profile structures and features in the subsurface. Pipe penetrating radar (PPR) is the in-pipe application of GPR.

Signal penetration depth is dependent on the dielectric properties of the pipe and the host material, and on the antenna frequency. Detectability of targets in the ground depends on their size, shape and orientation relative to the antennas, contrast with the host medium as well as external radio frequency noise and interferences. The penetration depth of high frequency antennas (1.0 GHz to 2.6 GHz) which are the most suitable for pipe investigations is on the order of 1 ft to 5 ft beyond the pipe wall, depending on the material of the pipe inspected. PPR can be used to detect pipe wall fractures, changes in material, reinforcement location and placement, and pipe wall thickness.

The resolution of PPR technology is primarily determined by the wavelength, but is also affected by other factors such as polarisation, dielectric contrast, signal attenuation, background noise, target geometry and target surface texture, all of which influence the reflected wave. Since the primary factor determining signal penetration is the conductivity of the soil, it is important to point out that PPR works where traditional "above ground" GPR does not.

The recorded raw data is processed in order to enhance anomalies at deeper levels. Frequency filtering is used to remove noise. By processing the data, more information is extracted as the weak and closely spaced events are enhanced. SewerVUE's proprietary RadART software package was used for applying different correction, gain and filter functions. The interpretation is then superimposed on the processed PPR profiles. Interpreted PPR profiles are correlated with the CCTV foldout views and presented together (Figure 1).

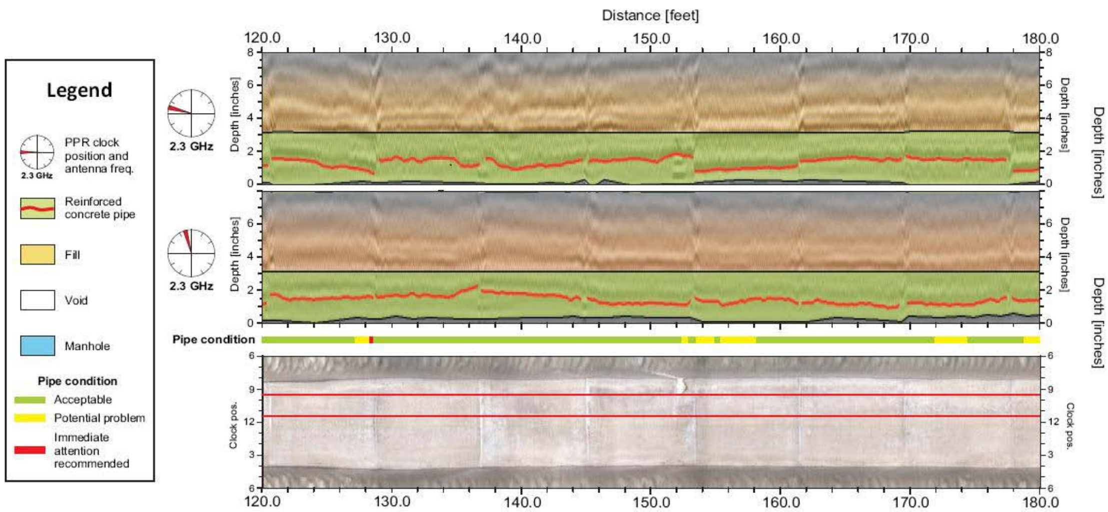

Figure 1. Robotic PPR data is displayed with the interpretation overlaid and correlated with the unfolded CCTV image.

SURVEY EQUIPMENT

The SewerVUE Surveyor is the first commercially available multi sensor inspection (MSI) robot that uses visual and quantitative technologies (CCTV, LIDAR, and PPR) to inspect underground pipes (Figure 2). This fourth generation PPR pipe inspection system is mounted on a rubber tracked robot and equipped with two high-frequency PPR antennae. The system used in Stockton, CA can be adjusted for 21 to 60-inch diameter pipes, the PPR antennae can be rotated between the nine and three o'clock positions. Radar data collection is obtained via two independent channels in both in and out directions, providing a continuous reading on pipe wall thickness and locating voids outside the pipe. CCTV data is recorded simultaneously and is used for correlation with PPR data collection.

Figure 2. The fourth generation SewerVUE Surveyor multi sensor inspection robot.

The sensors mounted on the robot take quantitative measurements of inside pipe walls. LIDAR technology employs a scanning laser to collect inside pipe geometric data which is then used to determine pipe wall variances from a manufactured pipe specification. LIDAR data is correlated with an onboard inertial navigation system (INS) that can accurately map the x, y, and z coordinates of the pipe without the need for external references. LIDAR and x, y, and z data collection was not part of the scope for this project.

CASE STUDY #1: PPR Inspection of a 36 inch lined RC pipe, Stockton, CA

The City of Stockton Department of Municipal Utilities commissioned to conduct a high-frequency pipe penetrating radar (PPR) survey to inspect sections of the Del Norte Trunk Sewer in Stockton, California (Figure 3). The Del Norte Trunk Sewer is a 36” reinforced concrete pipe (RCP) with a 0.7” thick fiberglass liner that reduced the inner diameter to approximately 32.5”. The total inspected length was 4264 feet, the inspection took place between November 6 and November 8, 2013. The objective of the PPR survey was to determine the condition of the approximately 55 years old lined RC pipe by mapping its wall thickness, rebar cover and detecting voids and/or other anomalies within or outside the pipe wall. The pipe experienced failures in the past and the fiberglass liner has at places also separated from the original RC pipe wall (Figure 4a and 4b).

This project’s PPR survey was completed using 1.6 and 2.3 GHz frequency antennae while the pipe remained in service. 2D line data were collected on the crown of the pipe. The PPR lines were located along the 10:00, 11:00, 12:00 and 1:00 o’clock positions inside the pipe.

Figure 3. Location of the Del Norte Trunk Sewer, Stockton, CA. Inspected segments are marked in red.

Both the 1.6 GHz and 2.3 GHz PPR data are of good quality. Signal penetration allowed analysis to a depth of 8 to 12 inches from the inside pipe wall surface. The most commonly used data displays are the two dimensional cross sections or the two dimensional depth slice (Figure 1). However, a user friendly data presentation that is readily understood and is faster to review by lay audience was developed for this report. The PPR inspection results are summarized on distance (feet) vs. pipe wall thickness and rebar cover (inches) graphs (Figure 5). These summary graphs are based on data extracted from the processed and interpreted individual PPR depth sections

Figure 4a. Deployment of the SewerVUE Surveyor at the Del Norte Trunk Sewer, Stockton, CA. 4.b separated liner.

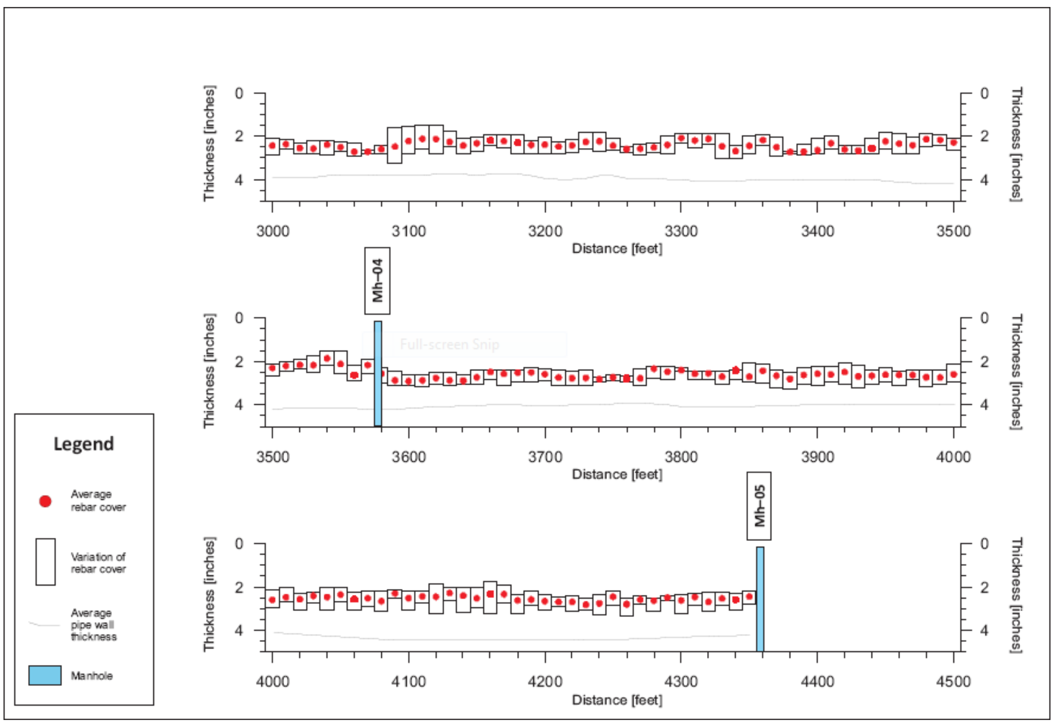

Figure 5. PPR results for the Del Norte Trunk Sewer, Stockton, CA. Pipe wall thickness is represented by a continuous black line. Change in rebar cover is represented by bar graphs showing rebar cover variations (min-max) for every 10 ft interval. Red dots mark average rebar cover for the same 10 ft interval.

The 4264 ft of PPR data from the Del Norte Trunk reinforced concrete sewer indicated sufficient rebar cover and no void type anomalies on any of the processed PPR profiles. Wall thickness including the grouted fibreglass liner was interpreted to be in the 3.9 to 4.5 inch range with little variation over the inspected length. The inspection identified localized liner failures but found the pipe in fair structural condition.

CASE STUDY #2: PPR Inspection of a 120 inch diameter brick lined sewer

The second case study took place in a 120 inch diameter brick lined sewer that was originally constructed in 1906. It is a combined sewer, that, during intensive rainfall events, fills to capacity, and conveys flows to a deep tunnel system and can also convey wet weather overflows to receiving waters.

In order to better inform the project team design on appropriate rehabilitation strategies, the knowledge of voids outside the sewer was important to rehabilitation methodology criteria. The engineering firm tasked with the design of rehabilitation commissioned SewerVUE to perform a non-destructive condition assessments before determining repair locations and methods. The primary purpose of the condition assessment was to locate and identify voids that may exist behind the brick lined pipe wall. This case study presents the methodology and results of the survey.

PPR instrumentation and field survey design

The inspection work was scheduled for the dry season for safety reasons and was completed in October 2014. A total of 2040 linear feet were inspected via manned entry. The inspection was limited to pipe penetrating radar (PPR) data collection at the 12, 9 and 3 o'clock positions.

The PPR survey was completed using a 1 GHz hand-held antenna system (Figure 6). This antenna frequency provided the optimal trade off between penetration and resolution and proved to be the most suitable in similar previous projects. A two person survey team carried out the work.

Figure 6. The 1 GHz hand held antenna that was used in the project.

The PPR antenna was placed on a custom made extension arm to ensure good antenna/pipe wall coupling. Since the 12 o'clock position was the most critical these lines were surveyed twice, generally in 50 ft increments. The 3 and 9 o'clock positions were surveyed once in 100 ft long sections. At the end of every (50 or 100 ft) line the data were saved and the file name was recorded.

Radar profiles are "upside down" relative to their actual orientations looking upward into the pipe crown, since PPR software customarily presents data looking downwards (Figure 7). The profiles have a depth scale in inches on the vertical axis, corresponding to about a 50 inches total depth of investigation. The depth scale of 0.33 ft/ns (0.1 m/ns) was derived from fitting a hyperbola over a diffraction pattern. This velocity fits with published and experimental data on masonry structures. All the collected PPR data are of high quality and rich in detail.

PPR data interpretation

The interpretation was based on the careful analysis of certain reflections that show the expected brick liner/fill interface in all of the depth slices in all directions. A given reflection was compared to the surrounding signal strength. A processed and interpreted depth profile is shown in Figure 7. The processed wiggle trace PPR profiles represent subtle variations of the reflected signal amplitude. Higher contrast represents higher amplitudes. When the higher amplitudes form a spatial extent, they are flagged as anomalies. These anomalies were coded and superimposed on the processed radar depth plots.

In order to illustrate how voids and other features were identified, a "surface inwards" interpretation approach is used here to discuss features in the profiles consecutively inwards from the first event at the surface. The first two arrivals are the airwave and the ground wave, the first two horizontal black bands on the profiles. These arrivals may on occasion mask shallow reflections. The hyperbolae represent the presence of anomalies that can be associated to targets, discontinuities or voids. The anomalies characterized by higher amplitudes of reflection and well-defined shapes are related to metallic elements randomly disposed.

The anomalies that are related to voids were marked considering a significant spatial extent of the hyperbolae. The size of the void was estimated trough migration (0.33 ft/ns) for a further categorization. Voids were characterized according to size: 2-12 inches, 12-36 inches and >36 inches. No voids larger than 36 inches in size have been identified.

The primary objective of the inspection was to identify voids larger than two inches in diameter outside the three courses of bricks. In addition to identifying void type anomalies, other anomalies were also marked and interpreted. These include: non-void type anomalies outside the pipe, which were interpreted as metallic and wood objects based on the signal pattern. Voids and possible damage zone within the three courses of bricks were identified together with the interpreted brick + mortar and fill interface.

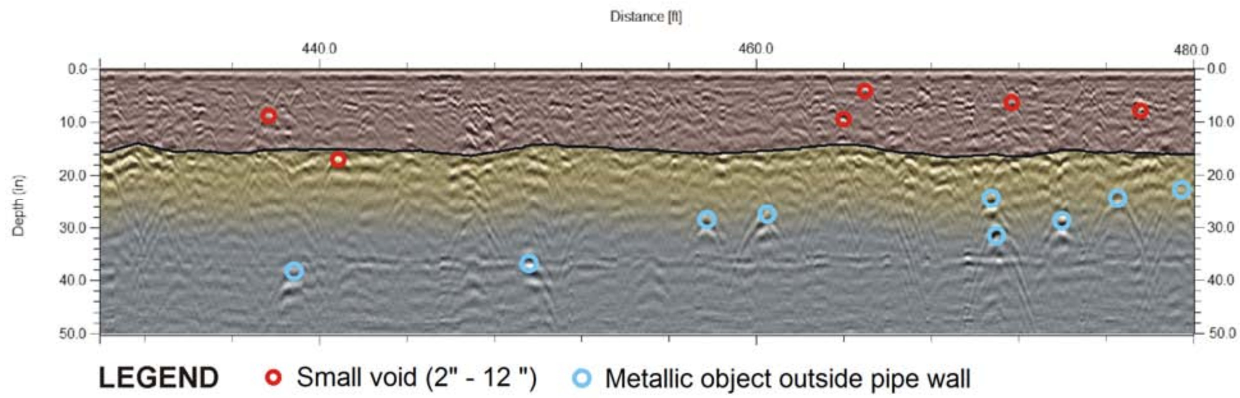

Figure 7. Interpreted PPR profile from a 120 inch diameter brick lined sewer.

PPR results

Areas of thicker void type reflections are sometimes seen along the liner-fill interface (Figure 7). These zones are conspicuous by their pronounced irregular “bright spot” anomalies with higher amplitude and lower frequency (i.e. wider banding) relative to their surroundings. Areas of interpreted voids and separations are marked on the interpreted sections.

Internal thicknesses of the void spaces are difficult to estimate as the bottoms of interpreted voids were rarely imaged directly. A void of more than a quarter wavelength in thickness (i.e. greater than about 1 in. to 1.25 in. in near saturated conditions) would be expected to generate additional bands; however, responses from the bottom of the voids were not separately identifiable. Size, orientation, infill and depth to the void are all significant. Void type anomalies were divided into three groups: small voids (2 -12 inch) medium voids (1 -3 feet) and voids larger than 3 feet. No voids larger than 3 feet were identified on any of the surveyed lines.

The three courses of bricks can be identified on some of the profiles some of the time (Figure 7). Well defined hyperbolic arches within the three courses of bricks were interpreted as small voids (Figure 7). Concentrated hyperbolic arch pattern indicate reflections and scattering from sides of individual bricks. This pattern may indicate missing mortar and these were identified as possible damaged zones.

Summary

A total 2040 ft of the sewer was surveyed with a hand held 1 GHz frequency PPR system. Signal penetration was between 20 to 40 inch depth, the data are of high quality. The most prominent feature on all the profiles is the pattern change often accompanied with a near horizontal, wavy interface at 16 ± 4 inch. This feature was interpreted as the brick liner/backfill interface (Figure 7). The observed anomalies were grouped into three categories in relation to this interface:

A) Anomalies within the three courses of bricks: well defined hyperbolic arches were interpreted as voids. A total of 220 small voids (2-12 in.) and 9 medium size (12-36 in.) voids were identified. A characteristic diffraction pattern from within the pipe wall was interpreted as pipe damage (e.g. missing mortar). 388 linear feet of possible pipe damage was found.

B) A total of 110 small (2-12 in.) and 27 medium size (12-36 in.) void type anomalies were found at the liner/fill interface. No voids larger than 36 inch were found on any of the surveyed clock positions.

C) Individual diffraction arches from within the fill (deeper than 18-20 in) most likely represent rocks, cobbles, boulders, timber and/or metallic construction debris and have no direct bearing on the structural condition of the pipe.

SUMMARY AND CONCLUSIONS

The key advantage of PPR is the unique ability to map pipe wall thickness and deterioration including voids outside the pipe, enabling accurate predictability of needed rehabilitation or the timing of replacement. Examples from a robotic and a manned entry project were used to illustrate how PPR can map remaining pipe wall thickness, rebar cover, grout placement and voids outside the pipe.

The robotic PPR case study that was conducted in Stockton, CA, in a 36" reinforced concrete pipe with a fiberglass liner confirmed 3.9 to 4.5 inch remaining wall thickness including the grouted fibreglass liner with little variation over the inspected length. Rebar cover appeared to be sufficient with no void type anomalies outside the pipe.

The 6,040 ft of high resolution PPR data from a 109 year old 120 inch diameter brick lined sewer revealed voids both outside and within the pipe wall and thus provided engineers the information needed to take the appropriate approach to rehabilitate the pipe.

With limited available funding and budget constraints becoming more prevalent, timing of rehabilitation and overall intelligent asset management is more critical than ever for municipalities and asset owners. Advanced pipe condition assessment technologies, including the SewerVUE PPR system, have demonstrated to be cost-effective, non-destructive methods that are able to help better refine structural condition and estimated remaining life of an interceptor, accurately determine overall severity of pipe degradation, as well as provide a basis for improved cost allocation and timing of rehabilitation efforts.

Case Study from Application of High-Resolution Ultra-Wideband Radar for QC/QA Analysis of Trenchless Pipe Rehabilitation and Pipeline Condition Assessment

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Abstract

Advanced condition assessment technologies for buried municipal pipeline applications have been gaining significant attention in recent years. This paper presents a case study from the recently completed pipeline condition assessment project employing Ultra Wideband (UWB) radar system integrated with CCTV camera. Recently, a storm drain was rehabilitated using the trenchless slip lining technique. Following rehabilitation, a non-destructive testing (NDT) based QA/QC analysis was performed to evaluate the quality of rehabilitation. Prior to the field test, laboratory based feasibility study was conducted using pipe specimen with artificially created defects. Results from both laboratory and field evaluations are presented in this paper. The results indicated the possibility of employing UWB radar as a tool for detecting quality control issues in the trenchless slip lining projects including grout blockage within the annular space, uniformity of grout thickness and proper alignment of liner pipe.

INTRODUCTION

Pipes that transport water (potable, sewer and storm water) constitute a significant portion of more than 6 million miles of buried pipes in the US (Sterling et al. 2009). These pipe networks are considered as national assets and play a critical role in smooth functioning of the society. It is well acknowledged that a significant portion of these pipes are nearing their design life and are suffering from increased rates of failure. Recent report card by the American Society of Civil Engineers (ASCE, 2013) highlighted the poor state of the US water infrastructure and challenges faced by the engineering community to maintain them. It was estimated that each year in the US approximately 240,000 water main breaks and 75,000 sanitary sewer overflows occur (EPA, 2010). The cost of maintaining and upgrading the aging water infrastructure is increasing as the rate of deterioration accelerates. Capital investments required for maintaining wastewater and storm water infrastructure alone is reported to be around $298 billion over the next 20 years. But the annual gap in the available capital to maintain water infrastructure is projected to reach $143 billion in 2040 from $54 billion in 2010 (ASCE, 2011). Because of this limited availability of resources, municipal engineers are constantly looking for cost effective non-destructive testing (NDT) technologies that could provide accurate condition assessment of buried pipes. Accurate assessment is critical to prioritize the rehabilitation work to pipe segments that require immediate attention and also to carry out QA/QC analysis post rehabilitation. While a number of NDT techniques have been reported in the literature for pipeline inspection, imaging through a typical sewer and storm water pipe to evaluate its condition and the surroundings in a rapid, non-contact fashion (with significant standoff distance between transducer and the pipe wall) is often complicated in a real world environment. Recently, a high resolution Ultra Wideband (UWB) radar for inspection of non-metallic pipes was introduced (Jaganathan et al. 2010). This paper presents a case study from the recently completed pipeline condition assessment project employing a novel pipeline inspection device that uses CCTV coupled with an UWB radar system (Future Scan).

OVERVIEW OF THE TECHNOLOGY

The system used in this case study consists of UWB pipe penetrating radar and CCTV camera integrated with a remotely controlled pipe crawling robot to simultaneously visualize the pipe's internal condition and 'see-through' the wall to evaluate the conditions outside. Knowing both internal and external conditions could allow for better interpretation of data. A detailed description of the UWB radar discussed here is provided elsewhere (Jaganathan at al. 2010, Taylor 1995). It is analogous to an impulse GPR where short bursts of electromagnetic energy is transmitted into the pipe wall and back-scattered signals are analyzed to detect targets. Targets are revealed as discontinuity in electrical properties such as permittivity and conductivity. The signal is transmitted (and received) using antennas that are held at a significant distance away from the pipe wall without need for physical contact or close proximity to the wall (Allouche et al. 2013). As a result, the robot is able to move relatively quickly at about 5 to 10 m/min unless an obstacle is encountered. Thus, the inspection could be completed rapidly without loss of productivity. Figure 1 provides a photograph of the robot and screenshot from the software interface. As seen in Figure 1, the software interface provides simultaneous images from the radar and the camera. The radar data is presented in B-scan (two dimensional space-time image of the pipe's cross-section along its length) and individual A-scan waveforms (signal strength vs time delay). The UWB radar system presented is limited to non-metallic pipes including concrete, plastic and VCP pipes that are about 45 cm to 90 cm in diameter. Recently, the UWB radar was employed to successfully detect soil voids outside HDPE pipe installed by the Horizontal Directional Drilling method (HDD), and to evaluate soil compaction outside a newly installed plastic culvert (Tony et al. 2015). In addition, ovality of the HDPE pipe was also estimated from radar data to within 2% accuracy compared with actual caliper measurements. In this paper, results from QA/QC analysis of a storm drainpipe rehabilitated by a trenchless method are presented.

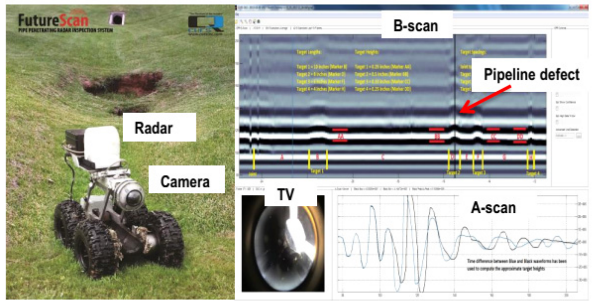

Figure1: (Left) – Photograph of the UWB radar system; (Right) – software interface showing simultaneous TV and radar images.

FIELD TEST DESCRIPTION

During 2013, Florida DoT contracted GraniteTech services, a subsidiary of CUES, Inc. to rehabilitate several sections of storm drain pipes that run under SR-19 LaBelle and Okeechobee, Fl. The deteriorated pipes were rehabilitated trenchlessly using slip lining method. In this method, a plastic liner pipe with diameter 5% to 10% less than the host pipe is inserted in using either "push" or "pull" technique (PPI, 2009). The annular space between the liner and the host pipe is then filled with low viscosity grout. The cured grout increases the structural stability of the liner by providing additional buckling resistance and also minimizes further deterioration of the host pipe. Some of the common problems encountered in a slip lining method include grout blockage within the annular space resulting in formation of air pockets and uneven grout thickness (Sullivan, 1992). These problems could potentially weaken the structure and create future leakages. However, the grouting material could not be just forced in place because of limited pressure rating of the liners used in practice. Thus, NDT based QA/QC procedure is required after grouting to ensure that no air pockets are formed and the grout thickness within the annular space is consistent.

During this project several storm drain host pipes made of reinforced concrete (RCP) were rehabilitated by slip lining them with plastic liner pipe, and the annular space was back-filled using cementitious "flowable fill" grout. The liner pipe (A-2000) used was made of PVC with a smooth internal wall and a corrugated outer wall. The internal diameter of the host pipes ranged from 45 cm to 92 cm, approximately. In this paper, QA/QC analysis performed inside two selected RCP pipes with an internal diameter of ~92 cm is reported. Figure 2 provides a schematic of the rehabilitated pipe's cross-section with corresponding dimensions given in Table 1. As seen in Figure 2, geometry of the cross-section involved was complex and the dielectric permittivity variation across the layers was also complicated. For example, factors that complicate the radar backscatter includes the corrugated wall of the liner pipe, presence of steel rebar within RCP and porosity of the grout. In an ideal project, QA/QC analysis could be carried out immediately following the rehabilitation process or while the grouting is in progress so that remediation action could be taken if any quality control issues were discovered. However, in this demonstration project, the condition assessment was performed several days after grouting. Figure 3 shows photograph of robot entering the rehabilitated storm drain.

As seen in Figure 3, there is significant standoff distance (~17 cm) between top surface of the radome cover and the pipe wall (shown as distance 'D' in Figure 2). During this project the following aspects were evaluated:

1. Ability to resolve individual layers of the composite structure (liner, host pipe and the intermediate grout layer) and estimate variation in grout thickness,
2. Detect air voids and other inconsistencies within the grout layer, and
3. Detect soil voids outside the host pipe.

Prior to the field test, a laboratory based feasibility study was undertaken to study some of the aspects mentioned above.

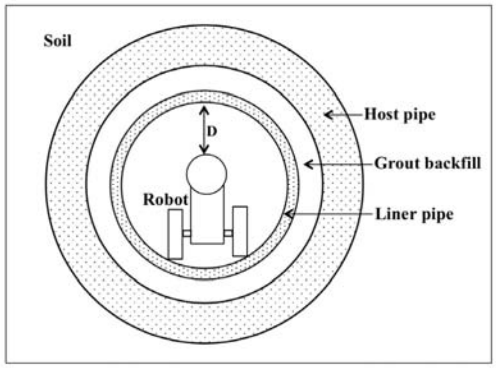

Figure 2: Schematic showing cross-section of the rehabilitated pipe segment.

Figure 3: Photograph showing the robot entering storm drain rehabilitated by slip lining method.

Table 1. Summary of different layers in the rehabilitated pipe.

Pipe	Material	Internal diameter (cm)	Thickness (cm)
Host pipe	RCP	92	9
Liner pipe	PVC	61	4
Grout layer	Cementitious grout	61	~7.5 (ideal)

LABORATORY BASED FEASIBILITY STUDY

Prior to field trial, radar analysis was carried out using a specimen constructed at the outdoor laboratory of the Trenchless Technology Center (TTC). The objective of this preliminary lab test was to evaluate if the radar has enough resolution to detect delamination and air pockets within the grout layer expected in a typical slip lining project. While the actual storm drainpipe to be inspected at Florida was made of RCP and buried underground, the lab specimen was constructed aboveground using a corrugated metal drainpipe 56 cm in diameter (and 6 m long) for convenience. A HDPE liner pipe (with 2.5 cm thick wall and 46 cm in diameter) was inserted into the metal pipe and the annular space was backfilled with cementitious grout. The thickness of the grout layer was about 7.5 cm. Figure 4 shows photographs of the specimen. While this specimen may not be an actual representation of the conditions seen at the Florida test site, it however simulates the conditions expected in many slip lining projects done in the industry. After the grout was allowed to set, artificial defects were introduced into it by carefully removing a small coupon from the metal pipe near the crown to give access to the grout layer. A small portion of the set grout was chipped away to simulate delamination (or an air pocket) not more than 0.5 cm in thickness, approximately. The removed coupon was carefully attached back such that no significant discontinuity was observed on the metal pipe. The specimen was subsequently inspected with the radar. The antennas were pointed toward the crown and moved along the entire pipe. While at present, CUES Inc. has developed a next generation system with an array of antennas for 360 degree coverage, at the time of testing, antennas had to be manually rotated to perform measurements at other angular positions.

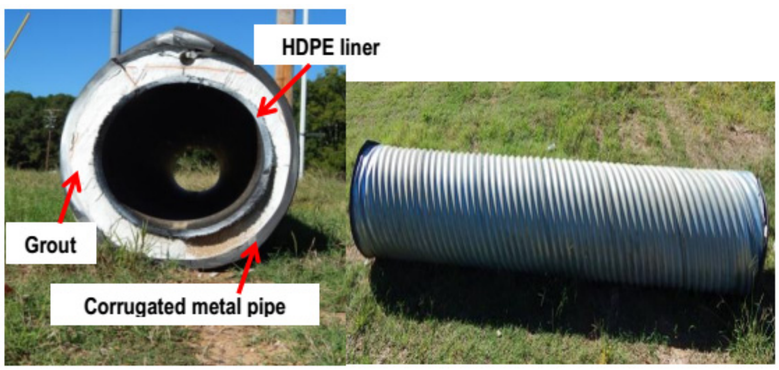

Figure 4: Photograph of slip lined corrugated steel pipe constructed for laboratory testing.

Figure 5 shows a screenshot of the radar data processing software. Three different plots are seen in Figure 5 - a) B-scan plot corresponding to the vertical cross-section near crown for entire length of the pipe (horizontal axis represent spatial increments along the pipe and vertical axis represent time delay in the received signal), b) portion of B-scan plot zoomed-in to show the details near delamination, and c) selected A-scan waveforms showing distinct signatures that are shifted in time indicating various radial interfaces present in the structure. Radar successfully detected the air pocket and a distinct signature was observed in the B-scan. Analysis of individual A-scan waveforms revealed that it is possible to resolve the HDPE and the grout layer. For example, the reflections from inner and outer wall of the HDPE pipe were seen clearly, and the interface between grout and the host pipe were also discriminated. Thus, the laboratory test successfully demonstrated the ability of the radar to detect hidden air pockets and resolve the grout layer thickness. This capability could be used to estimate thickness variations and other quality control problems that could occur in a slip lining project. Also, the laboratory experiments provided baseline measurements for comparison against the field data.

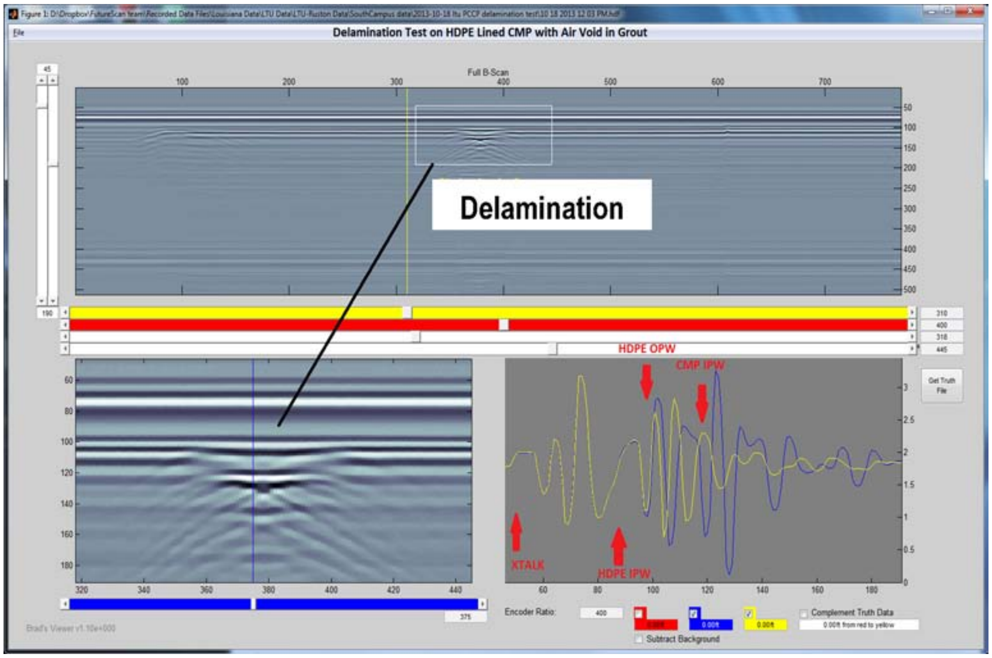

Figure 5: Screenshot from radar analysis: (top) 2D B-scan image of the pipe's vertical cross-section near crown along its entire length; (left bottom) zoomed in window showing delamination; (right bottom) A-scan waveforms (signal strength vs. time delay) showing interfaces between HDPE, grout and the metal pipe.

RESULTS FROM FIELD TRIAL

After successful completion of the lab test, field trials were carried out during July 2013. Two RCP storm drainpipes (each ~55 m in length) running parallel to each other (shown in Figure 3) were inspected. Field trials took place several days after grouting and this allowed enough time for the grout to set, at least partially. Portland cement based "flowable fill" grout was used in this project. Dielectric permittivity of such a material is influenced by several factors including frequency of the signal, its mineral and moisture contents (Jaganathan et al. 2007). Since the grout's moisture content gradually varies over time, its permittivity would also change and this would affect the amount of reflection and transmission of radar signals through it. Thus, the radar inspection could be influenced by whether the grout is in a "flowable" state or set, at least partially, with reduced moisture levels. As long as there is a significant difference in permittivity between adjacent layers it would be possible to discriminate them. While the geometry of each layer involved is complicated, for a simple case of normal incidence of plane wave, the co-efficient of reflection (Γ) and transmission (T) at an interface between two dielectric layers with permittivity ε_1 and ε_2 are given by the following equations (Balanis, 2012):

$$\Gamma = \frac{\sqrt{\varepsilon_1} - \sqrt{\varepsilon_2}}{\sqrt{\varepsilon_1} + \sqrt{\varepsilon_{21}}}$$
$$\mathrm{T} = \frac{2\sqrt{\varepsilon_1}}{\sqrt{\varepsilon_1} + \sqrt{\varepsilon_2}}$$

Before grouting, the liner pipe was not secured in place and it could have floated up under pressure during grout injection leading to improper alignment and uneven annular space between the pipes. Thus, estimating the uniformity of grout layer, detecting air pockets and pipe alignment were of interest in this QA/QC analysis. The radar plots for each of the two parallel lines inspected (shown in Figure 3) are given in Figure 6 as two individual screenshots. Full length B-scan plots shown in Figure 6 represent the entire pipe length (~55 m). In Figure 7, a selected window of B-scan is presented to reveal finer details. Following observations were made during post processing of the collected data:

a) The pipe joints in A-2000 and RCP host pipes appeared distinctly in the B-scan at regular intervals in the expected places (seen in Figure 7). Their location was in agreement with prior knowledge about the pipes (one of the lines had ~0.9 m long RCP and ~6 m long A-2000 segments, while the other had ~3 m long A-2000 segments). The pipe joints detected by the radar matched well with CCTV observations. The sensor fusion capabilities provided by the UWB radar lead to better interpretation of radar data and minimized the uncertainties. This feature is expected to be very useful, especially, when the field conditions are not known in advance.

b) The interfaces between different layers present in the rehabilitated pipe were clearly discriminated. For example, interface between air and the inner wall of A-2000 pipe appeared distinctly as a constant line. This was expected because the robot travelled smoothly inside the new pipe maintaining a consistent separation distance between antenna and the inner pipe wall.

c) The interface between grout layer and adjacent pipes were visible distinctly. Using this, grout layer was resolved, and its relative thickness was estimated. The time-of-flight calculations were performed using the A-scan signals (shown in Figure 8) in order to calculate thickness of each layer (Falorni et al. 2007). This calculation requires that the wave velocity through each layer is known which again depend up on their dielectric properties. As mentioned previously, the dielectric permittivity of grout layer depend on moisture content, and it was an unknown value. However, through calibration and by assuming the grout layer to be homogeneous, relatively variation in its thickness was established with reasonable accuracy. As seen in Figure 7, thickness of the grout layer showed significant variations along the pipe. By assuming the grout's permittivity to be constant over the entire pipe, its thickness was estimated to vary by more than 50% (from about 7.5 cm which was the maximum possible height of annular space to about ~3 cm at its thinnest section). More importantly, it was established that the entire annular space was filled with grout and no significant air-pockets were observed. B-scan signature of air pockets collected during the laboratory test was valuable during this analysis.

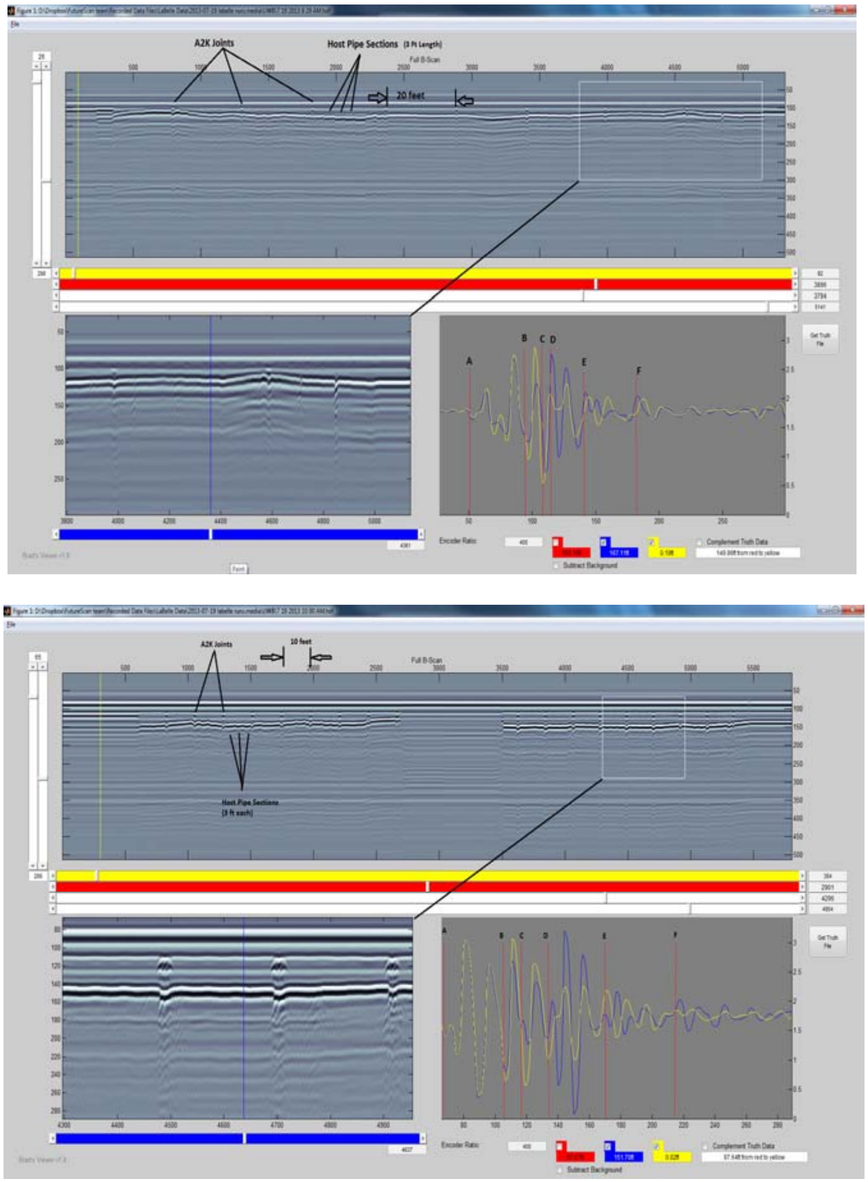

Figure 6. Screenshots from radar analysis of the two rehabilitated storm drain pipes (top and bottom images correspond to each of the two individual pipes seen in Figure 3 that run parallel to each other); Full length B-scan plots shown at the top of these two images represent the entire pipe length inspected.

d) The inner wall of the RCP host pipe was also discriminated clearly. The pipe-soil interface was also visible for the most part. But, in certain locations, the outer pipe wall was not clearly distinguished. This could be attributed to several factors including: i) only a small portion of the energy reached the host pipe because of signal losses that occurred in the two preceding layers (liner and grout), and ii) under certain conditions the permittivity of concrete and the surrounding soil could be very close resulting in weak reflections. However, the rebars buried within the host pipe were detected. No major soil voids outside the host pipe were detected. Thus, the UWB radar was able to assess the host pipe's condition seeing through the two extra layers added after rehabilitation. Qualitatively better data could be expected from the host pipe if the radar survey was conducted prior to slip lining.

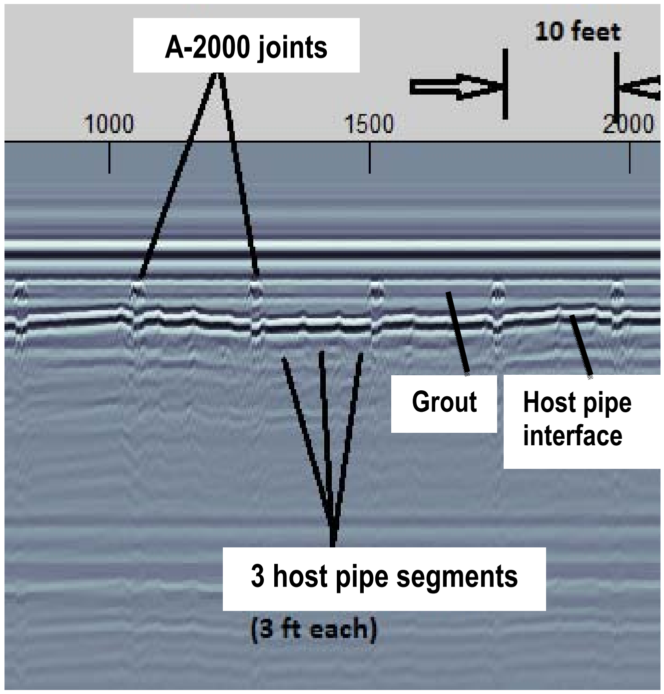

Figure 7. Selected window from the B-scan plot showing A-2000 pipe joints at ~3 m intervals, ~0.9 m long RCP host pipe segments (separated by faint hyperbolas seen at the joints) and the grout layer.

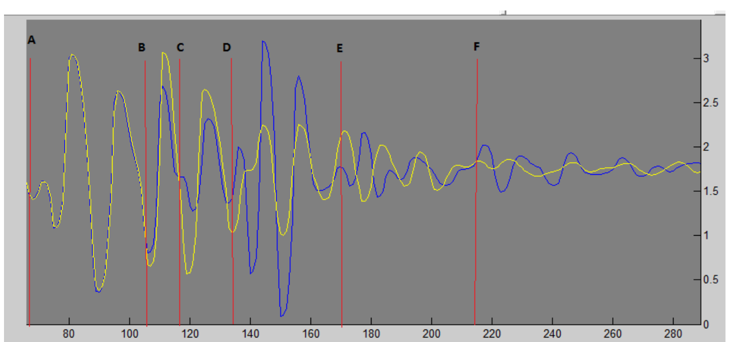

Figure 8. Selected A-scan signals from two different lengths (markings A to F represent the reflected signal from different layers of a given cross section).

CONCLUSIONS

The following are the conclusions from this case study:

1. The ability of the UWB radar to carry out high-resolution QA/QC analysis of buried pipes rehabilitated by slip lining technique was demonstrated,

2. The UWB radar was able to resolve individual layers in the rehabilitated pipe, and assess the quality of grouting and proper alignment of liner pipe. The radar was also able to partially discriminate the host pipe by seeing through the extra layers added during rehabilitation,
3. The sensor fusion obtained by integrating the UWB radar with CCTV allowed for better interpretation of the radar data, and
4. The laboratory and field trials indicated that the UWB radar could possibly be employed as a tool by municipal engineers to perform condition assessment as well as conduct QA/QC analysis post slip lining rehabilitation of buried sewer and storm water pipes.

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Drinking Water Pipelines Defect Coding System

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Abstract

Many regulatory agencies around the world require their municipalities and water utilities to routinely assess and report the condition of their water and wastewater assets. Condition assessment standards and protocols exist for wastewater and gas pipelines as well as for other civil infrastructure systems such as pavements, bridges and buildings. However, no standard defect coding and condition grading protocols exist for potable water pipelines. The development of a standard coding and condition classification system for water distribution mains is challenging because there is no single inspection technology that can detect and characterize all pipeline flaws, defects and features. Therefore, the codes and classification protocol ought to be independent of inspection technology. This paper introduces the development of a standard defect coding system for drinking water distribution pipelines. Common anomalies, defects and failure modes for metallic (cast iron, ductile iron, and steel), plastic (PVC and PE), and asbestos cement water mains are briefly discussed. The paper also highlights the challenges related to water main condition assessment and discusses existing specifications and standards from gas and petroleum industry that can be adapted by the water industry to develop an objective condition assessment protocol for water mains. The proposed standard defect coding system is being developed with the support of Water Research Foundation and in collaboration with over a dozen municipalities and water utilities from Canada and the USA, as well as major technology providers and international water experts. It will provide a common nomenclature and language for water main defects and features. Other benefits will include facilitation of effective and efficient asset management, support for benchmarking and establishment of minimum acceptable condition levels or levels of service, and improved operation, maintenance and renewal of water distribution systems.

INTRODUCTION

Municipalities and water utilities around the world strive to provide their customers with safe and reliable water supply at adequate pressure. Historically, buried water pipelines remained out of sight and neglected for a long time (Grigg, 2012; Jung et al., 2014). Furthermore, water distribution pipes in many jurisdictions have reached the end of their service life (Rehan et al., 2013). This has resulted in water safety and quality issues and accelerated the water pipelines degradation, failures and breaks. The ensuing water pipeline failures, water quality concerns, and increased operational and maintenance costs triggered new regulations that required the municipalities and water utilities to assess the condition of their water infrastructure and develop short- and long-term financial sustainability plans. Rehan et al. (2013) discuss the performance and condition assessment reporting, and financial sustainability related statutory requirements and regulations (e.g., Safe Drinking Water Act, 2002; Sustainable Water and Sewage Systems Act, 2002; Ontario Regulation 453/07; PSAB 3150; and Water Opportunities and Water Conservation Act, 2011) in Ontario, Canada. The US National Research Council (NRC, 2006) provides an overview of federal, state and local regulations and statutes related to the performance requirement of drinking water distribution systems in the United States.

Whereas standard condition assessment protocols exist for oil and gas pipelines, industrial/plant piping, and wastewater collection pipelines, no standard condition classification system exists for drinking water pipelines. Since the 1960s, extensive research to understand pipelines failure mechanism and modes resulted in "fitness-for-service" and "fitness-for-purpose" methods (described in more detail in subsequent sections). These methods have been modified for quantitative assessment of pipeline integrity in the petroleum and plant piping industries. Similarly, WRc's Manual of Sewer Condition Classification and Sewerage Rehabilitation Manual gained widespread acceptance for condition assessment and renewal planning of wastewater collection networks. However, no systematic condition classification protocol exists for water mains. Various reasons contributed to this situation, including lack of adequate funding and complexity of water distribution systems, pipelines are buried underground with few (if any) access points, pipes consist of a variety of materials, which in turn resulted in a variety of inspection technologies and condition assessment propositions. Limited work has been done on synthesizing the evaluation of various types of defects and resulting failures in water distribution pipelines.

In 2014, recognizing the need for a standard condition assessment protocol for water distribution pipelines, the Water Research Foundation initiated a project entitled "Potable Water Pipeline Defect Condition Rating". The project objective is to develop the framework and contents of a standard defect coding system for potable water distribution pipelines. The three main components of the proposed framework include: (1) a simplified, risk-based approach for preliminary prioritization of water mains for condition assessment, renewal, etc.; (2) defect coding and water main condition classification system; and (3) decision support system to determine, based on water main condition, when to rehabilitate, replace or to follow up with more rigorous condition assessment technique.

Municipalities and water utilities will benefit from the standardized approach for condition assessment and management of buried water mains. The defect coding system, an important component of this standardized approach, will serve as the basis upon which inspection-discerned distress indicators will be interpreted into pipe condition rating. The benefits include: (1) industry standard terminology, inspection surveys, and data format; (2) contractors' quality assurance and quality control using certification programs; (3) water main inspection technology and software vendors using standard data format resulting in data portability; (4) standardized critically analysis enabling different water utilities to communicate using the same language; (5) development of data benchmarking performance indicators; (6) cost savings; and (7) decreased subjectivity, and high quality, consistent data.

PIPELINES CONDITION ASSESSMENT PROTOCOLS IN WASTEWATER AND PETROLEUM INDUSTRIES

Sewer Inspection Protocols

In the 1970s, the Water Research Centre (WRc) and the Transport and Road Research Laboratory initiated the development of a methodology for describing internal condition of sewers in the UK (Thornhill and Wildbore, 2005). The preliminary or "Embryonic Codes" for sewer defects were developed in 1978 and formed the basis for the first edition of Manual of Sewer Condition Classification (MSCC1) that was published in 1980. The project to develop the Sewerage Rehabilitation Manual (SRM) started in 1978 and the first edition of SRM was published in 1983. As experience and knowledge grew, the MSCC1 was revised and improved, and subsequent editions MSCC2, MSCC3 and MSCC4 were released in 1988, 1993, and 2004, respectively. The MSCC4 includes condition codes for manholes and inspection chambers. The Sewerage Rehabilitation Manual was revised and new developments and improvements were incorporated into subsequent editions. They were published as SRM2, SRM3 and SRM4 in 1990, 1994, and 2001, respectively.

The simplicity and practicality of MSCC and SRM resulted in their widespread acceptability and adaptation by water utilities, and municipalities around the world. The Canadian market adopted the MSCC3 in the 1990s. In an attempt to standardize sewer defect coding and to remove subjectivity in condition assessment, the North American Association of Pipelines Inspectors (now disbanded) trained CCTV camera operators, contractors and municipal personnel across Canada. In the United States, the National Association of Sewer Service Companies (NASSCO) contracted with WRc to develop Pipeline Assessment and Certification Program (PACP). The NASSCO's PACP was based on MSCC3/EN: 13508: Part 2. In 2011, the Canadian Standards Association (CSA) and NASSCO jointly introduced PACP v6 to the Canadian market for visual inspection of sewers using CCTV cameras. The PACP v6 included CSA Plus 4012-10 Visual Sewer Inspection Technical Guide that includes guidance on defect severity and extent and provides a framework for mapping and organizing PACP defect codes into pipe failure modes.

WRc's MSCC3, the most widely used manual for sewer condition classification, includes a total of 69 codes for structural and operational defects as well as codes for construction and miscellaneous features. The defect scoring scheme, based on severity and supplementary information about soil, surcharging and backfill/pipe casing, contained in the fourth edition of Sewerage Rehabilitation Manual (SRM) has been used extensively for computing structural and operational internal condition grades and structural performance grades. The SRM was changed to Sewerage Risk Management and the first edition was published in 2006. The current, revised SRM was released online in 2014.

Petroleum Pipelines and Industrial Piping Condition Assessment

In the 1960s and 70s, several pipelines failures and resulting damages, including loss of human life, necessitated the development of "fitness-for-service" and "fitness-for-purpose" assessment methods (Escoe, 2006). According to Cosham et al. (2007), fitness-for-purpose methods combine fracture mechanics with pipe inspection, and involve quantitative, engineering critical assessment of defects in an object (e.g., pipeline, structure). The defects are evaluated and the system performance is assessed against certain standards and specifications. In the

1960s, Battelle Memorial Institute of Columbus, Ohio, along with a gas transmission pipeline company carried out tests to investigate the relationship between degree of corrosion (defect size) and internal pressure that would cause a leak or rupture (ASME, 2012). In the 1970s, the American Gas Association advanced the research to develop relationship between pipe strength and defect size. Over the last 50 years, numerous full scale studies investigated pipeline defects and assessed their significance in making repair, rehabilitation or replacement decisions (Cosham et al., 2007). This has resulted in numerous best practice guidelines, specifications and standards to carry out pressure pipelines inspections, to interpret and report survey results in a consistent manner, to carry out pipelines renewal, and to ensure safety and integrity of pipelines. Two well-known standards include API 579: Recommended Practice for Fitness-for-Service and BS 7910: Guide to Methods for Assessing the Acceptability of Flaws in Metallic Structures (Matthews, 2004). The Pipeline Defect Assessment Manual (PDAM), an on-going joint industry project that commenced in 1999, proivdes information on the best practices and techniques to assess pipelines defects and to ensure pipelines safety and integrity (Cosham et al., 2007). The American Peterolium Institute's API: 1163 In-line Inspection Systems Qualification Standard (API, 2005) incorporates Pipelines Operators Forum's "Specifications and Requirements for Intelligent Pig Inspection of Pipelines" (POF, 2009). Together, these two documents set the standards for consistent pipelines condition assessment, selection and proper application of suitable inpsection technology, common nomencalure for pipelines anomalies, desired accuracy, data QA/QC and consistency, and reporting of inspection results.

WATER MAIN CONDITION CLASSFICATION FRAMEWORK

Key Considerations and Challenges

A state-of-practice review revealed the following key considerations and challenges to develop a standard water main condition classification system.

1. Many municipalities and water utilities prefer and/or practice a reactive approach to manage their water distribution networks and do not carry out condition assessment of water distribution systems on regular basis. A scarcity therefore exists of condition assessment data on water pipelines. Furthermore, many jurisdictions have incomplete inventories and unreliable information on water pipelines age, size, location, materials, and maintenance and rehabilitation history.
2. High inspection costs may be prohibitive. Therefore, depending on the availability of funds, market conditions and inspection budget, selective inspection of drinking water systems is carried out in some jurisdictions.
3. Unlike wastewater pipelines condition assessment, where CCTV is widely used for visual inspection of inside surface of pipe, there is no single technology that is appropriate for the inspection of all the types of water pipelines in existence (ductile iron, cast iron, steel, plastic, AC have different types of defects and failure mechanisms).
4. Limited access to buried water pipelines creates additional challenges for many in-line as well as external inspection technologies.

Figure 1 provides an overview of the proposed framework. It is to note that only metallic (cast iron, ductile iron, and steel), plastic (PVC and PE) and asbestos cement water pipes are considered in this project. The important components of the proposed framework are discussed below.

Step 1: Water Pipes Inventory – Data Requirements

It is important to note that utilities have varying level of data, and there is general paucity of data on potable water networks. Therefore, three levels of data requirements, namely minimal, adequate, and comprehensive are defined and suitable attributes related to asset (inventory), operational/performance and life-cycle event are specified (Kleiner and Colombo, 2014).

Level 1 – minimal data: Asset Inventory (e.g., pipe ID, material, diameter, and length); Operational/Performance (e.g., failure, repair, and renewal); Life-cycle Event (e.g., installation date; failure, repair, and replacement history/dates).

Level 2 – adequate data: Level 1 data plus Asset Inventory (e.g., pipe wall thickness, depth, pipe manufacturer and process, soil/bedding, geographic information, lining/coating information, valve type/location); Operational/Performance (e.g., operating and transient pressures, cathodic protection, valve inspection/repair/renewal history); Life-cycle Event (e.g., cathodic protection date, valve inspection/repair/renewal date, inspection history).

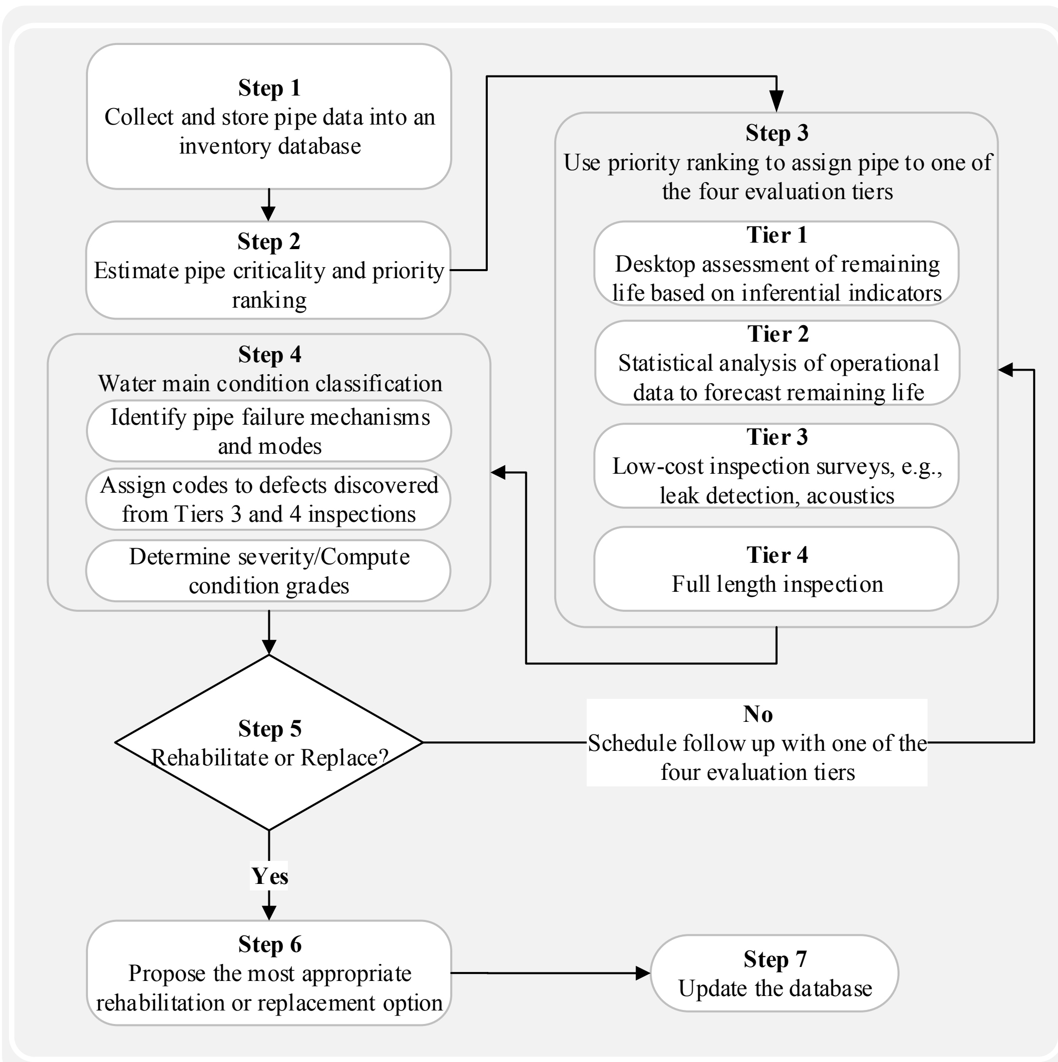

Figure 1: Overall Framework for Water Main Condition Classification ***(Copyright Water Research foundation, 2015; used with permission)***

Level 3 – comprehensive data: Level 2 data plus Asset Inventory (e.g., proximity data, ID and location of shutoff valves, soil characteristics, groundwater table, ground movements, traffic information/loads, pipe/soil potential survey, soil resistivity); Operational/Performance (e.g., customers complaints, water quality, out of service time, cost data, damage to adjacent properties, shutdown time, photos, pipe/soil samples); Life-cycle Event (e.g., inspection date, rehab/replacement activities, damage/failure).

Step 2: Pipelines Prioritization for Inspection

The condition of a water main can be discerned through observable or measurable signs (or 'distress indicators') that point to structural deficiency or damage, as well as, 'inferential indicators' that point to the potential existence of deterioration mechanisms. Whereas distress indicators (e.g., cracks, corrosion pits, inclusions) are obtained through actual pipe inspection, inferential indicators (e.g., soil corrosivity, pressure transients, and overhead traffic) are discerned through information about the pipe general environment and operational conditions. Using a simplified risk-based approach, pipes are ranked based on failure risk, i.e., failure likelihood coupled with failure consequence (also referred to as pipe criticality and includes direct, indirect and social cost of failure).

Step 3: Water Pipelines Preliminary Categorization

Following the prioritization process, the pipes are categorized into four tiers for detailed condition assessment as specified below:

Tier 1 is quick, simple assessment through desktop process using inferential indicators for the lowest criticality pipes.

Tier 2 is more detailed assessment, employing statistical analysis to forecast remaining life, for higher criticality pipes for which some historical failure data are available.

Tier 3 involves low cost inspection surveys (e.g., leak detection, acoustics) for: (a) critical pipes whose condition is rather uncertain for which some concern exists but expensive full-length inspection cannot yet be justified; and (b) lower criticality pipes with a relatively long remaining life but strong evidence of past leakage.

Tier 4 is the most detailed full length assessment applied to high-criticality pipes using visual or Non-Destructive Evaluation (NDE) means.

With this four-tier approach, the economics of using a given inspection technology depends on the cost of failure, cost of pipe renewal, cost of inspection technology, probability of detection of the technology as well probability of false positives.

Water Pipelines Inspection for Tier 3 and Tier 4 Pipes

Inspection or condition assessment survey refers to one-time measurement(s) and involves process(s) or combination of activities to establish the condition of an object or system at a given point in time (Hunaidi and Bracken, 2007; Roberge, 2007). Some of the existing water main inspection technologies and methods include visual inspection, electromagnetic methods (e.g., magnetic flux leakage, eddy current), acoustic methods, and ultrasonic testing. There have been new developments and rapid improvements to water main inspection techniques in recent years. The choice of inspection method mainly depends on the availability of funds, purpose of inspection and desired level of accuracy. Furthermore, factors such as pipe material, pipe length and geometric features, geographic conditions, and operating environment also dictate the choice of condition assessment techniques. In some cases a combination of multiple-mode inspections and combination of inspection techniques are needed for more accurate measurements (Roberge, 2007). Liu and Kleiner (2013) and Lillie et al. (2004) provide an excellent account of existing and emerging water main inspection techniques, their working principles, and limitations. Table 1 provides a summary of condition assessment technologies applicable to metallic, plastic and asbestos cement water mains. For AC water mains, Hu et al. (2013) discuss other suitable techniques that include phenolphthalein test and scanning electron microscopy – energy dispersive spectroscopy analysis.

Following Level 4 inspection, all discerned distress indicators (flaws and defects) are coded as discussed briefly in the following section.

Step 4: Water Main Condition Classification

Identify Pipe Failure Mechanisms and Modes

Failure mechanisms are the deterioration processes that lead to failure, whereas, failure modes refer “to the actual manner in which” pipe fail (Makar et al., 2000). The condition and performance of water mains deteriorates over time because of a number of factors and complex physical processes (Liu et al., 2012). They include, for example, changes in operational conditions; environmental degradation and wear; mechanical, biological or chemical degradation processes; accidental or intentional interference; flaws during pipe manufacturing process; poor choice of pipe material; faulty design; poor installation; and natural events (Glisic, 2014; Liu et al., 2012). These factors result in flaws and defects that compromise structural integrity and operational performance of water distribution systems and result in water main breaks and failures.

Metallic Water Mains (Cast Iron, Ductile Iron, and Steel): Metallic water mains include pipelines made from iron alloys such as cast iron, ductile iron and steel. Cast and ductile iron, and steel are produced by altering the carbon content and further processing of pure iron which has low strength and rapid oxidation properties. *Cast iron* has carbon content greater than 2%. The grey cast iron has lamellar (small plate or flake) graphite. *Ductile iron* is a cast iron product obtained by adding magnesium to molten low-sulfur based iron which converts free graphite into spheroids and imparts ductility and strength (Mays, 2010). *Steel* is an iron alloy with less than

Table 1: Summary of Condition Assessment Technologies Applicable to Different Pipe Materials ***(adapted from Liu et al. (2012))***

Technology	Metallic Pipes	Plastic Pipes	Asbestos
	DI; CI; Steel	PVC PE	Cement
Visual Inspection	☑	?	☑
Pit Depth Measurement	☑	-	-
Electromagnetic Inspection	☑	-	-
Ultrasonic Testing	☑	?	-
Acoustic Inspection	☑	☑	☑

☑ : available; ?: may or may not work

2% carbon content. For an overview of metallurgical and mechanical properties and chemical composition of metallic water mains, please refer to Makar and Rajani (2000) and Mays (2000).

All metallic pipes can be subject to external and internal corrosion (both general and pitting), manufacturing flaws, excessive loading, improper design and installation, and human error. However, cast iron is a brittle material whereas ductile iron and steel are ductile resulting in different predominant failure modes. For cast iron water mains, failure modes can include (Makar et al., 2001): blow out holes, circumferential and longitudinal cracking, bell splitting, bell shearing, and spiral cracking. The predominant failure mode in ductile iron water mains is corrosion pitting and perforation with very few observed circumferential or longitudinal splits, (Rajani et al., 2011)). For steel water pipelines, external pitting corrosion and perforation/pinholes, and defective joints are predominant failure modes. Folkman (2012) reported circumferential and longitudinal cracks in steel water pipelines in some Canadian and US jurisdictions.

Plastic Pipes (PVC and PE): Plastics are polymers (thermoplasts) whose properties have been modified with additives to make them suitable for desired applications (Farshad, 2006). The physical, mechanical, thermal and chemical properties of plastic pipes are time and temperature dependent (Farshad, 2006; Lewis, 2000). Factors, such as poor pipe material selection, defects and flaws during the manufacturing process, poor design, and improper handling and installation can result in various types of flaws, defects and failures. The predominant failure modes include cracking due to scratches/surface damage, voids and inclusions (Liu et al., 2012). The unplasticized PVC is brittle and susceptible to porosity and degradation from poor processing and cyclic fatigue stress (Farshad, 2006; Lewis, 2000).

Asbestos Cement (AC): AC pipe was first introduced to the North American market in the late 1920s, and was used extensively for potable water distribution from 1940s to early 1970s (Hu et al., 2010). Mordak and Wheeler (1988) found that low pH and low alkalinity water cause significant deterioration to AC water pipes. According to Hu et al. (2010), two main failure mechanisms for AC pipe include lime leaching and sulphate attack.

Pipelines Defects and Features Identification

Pipelines defect and features identification involves detecting, sizing, characterizing, and documenting defects and features following a condition assessment survey. Figure 2 provides an example of sizing metal loss in terms of length, L, and width, W, of anomalies. Figure 3 illustrates the start, end, depth and clock position of metal loss, as well as reference wall thickness. The detection capability and accuracy of inspection tools and techniques are influenced by a number of factors, such as operator skill in technology application and data interpretation, calibration, technology limitations, and operational conditions (Bubenik, 2014; Roberge, 2007). Therefore, it is important to include validation and performance measures of the inspection tool and technique used and to carry out verification measurements. The American Petroleum Institute (API, 2005) and Roberge (2007) suggest the following key performance measures for inspection tools:

Detection: “ability to find pipeline features and anomalies.”

Identification: ability to correctly identifying the detected anomaly and differentiating it from other anomalies.

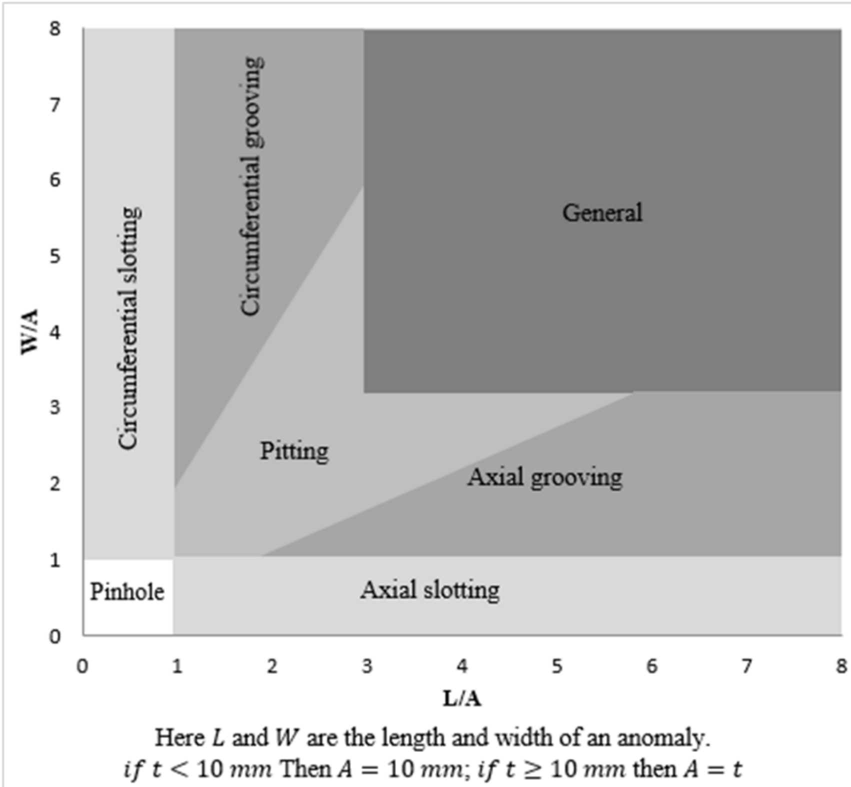

Figure 2: Sizing Chart for Metal Loss Anomalies ***(adapted from POF (2009))***

Sizing Accuracy: ability to correctly find size (length, width, depth) of various types of anomalies.

Location: ability to find the extent (start, end) and orientation of pipe features and various types of anomalies.

According to API (2005), when reported condition assessment results are inconsistent or do not match with historic data (for cases when prior data is available), verification digs are recommended. The reported inspection results are compared with the verification measurement to validate inspection information.

Assign Codes and Document Water Main Features and Defects

Unlike wastewater pipeline inspection where CCTV (closed circuity television) is almost invariably used, a wide variety of inspection technologies exist for water main inspection. It is to note that no single technology can detect and identify all pipeline flaws, defects and features. Therefore, the proposed codes for water main flaws, defects and features need to be independent of inspection technology. Thus, some codes may not be applicable for a particular inspection technology. For example, MFL-SR (Magnetic Flux Leakage – Standard Resolution) metal loss tool cannot detect circumferential crack (coded as CC) or axial crack (coded as CL) (API 1160, 2013). Therefore, codes CC and CL are not applicable for MFL-SR inspection tool. Additional information including inspection technology type (e.g., electromagnetic flux leakage, ultrasonic testing, visual inspection, acoustics, etc.); equipment make, model, and sensors; and limitations and performance thresholds will be part of the reporting requirements. Furthermore, descriptions of flaws, defects and features will include the applicable pipe materials. A training/certification program can be developed to enhance the knowledge and skills of inspection operators to use appropriate codes for pipeline flaws, defect and features. The following paragraphs provide preliminary information about header section and codes for pipe flaws, defects and featrues.

Header Section: This includes information related to, for example, Client; Project/Contract; Geographic/Location; Inspector/Surveyor; Contractor(s); Date/Time of survey; Pipeline ID; Reference Start/End of Survey; Survey Length; Direction; Pipe Material; Pipe Wall Thickness; Pipe Diameter; Pipe Depth; Lining/Coasting; Soil Information; and Survey Purpose.

Additionally, the following performance information on condition assessment technology is included:

- Detection thresholds for various flaws and defects. For example, for corrosion, minimum depth, width, length and orientation should be specified.
- Probability of identification
- Probability of detection
- Ideal and prevalent operating conditions

Codes for Defects and Features: Pipeline flaws and defects can be broadly classified into manufacturing, construction, and operational. Additionally, there can be coating and lining

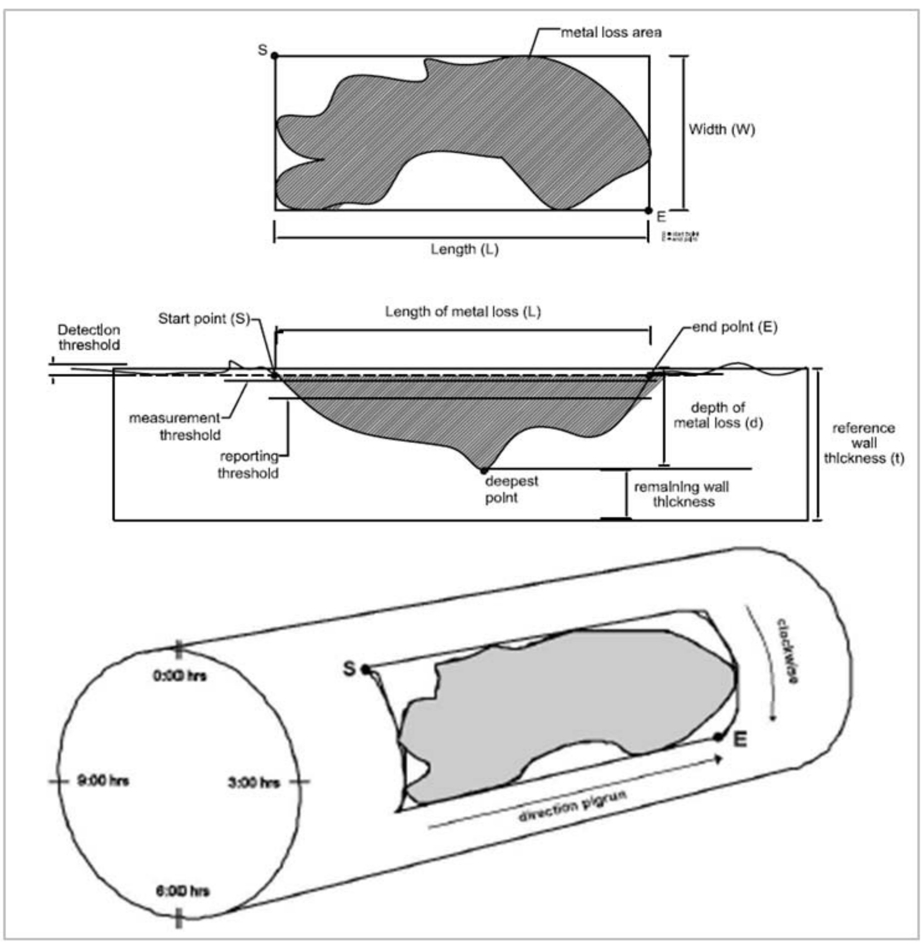

Figure 3: Metal Loss Measurements *(adapted from POF (2009))*

Table 2: An Example of Water Pipelines Selected Flaws and Defect (API, 2005; POF, 2009; and WRc, 1993)

Class	Defect/Feature and Code			Class	Defect/Feature and Code	
Structural Defects	Metal Loss	General	CG	Structural Defects	Buckle	B
		Axial Grooving	CAG		Dent/Deformation	D
		Axial Slotting	CAS		Joint Displacement	JD
		Circumferential Grooving	CCG		Surface Damage	SD
		Circumferential Slotting	CCS		Lining Defect	LD
		Pitting	CP	Construction Features	Connection	CN
		Pinhole	CPH		Connection Intruding	CNI
	Tuberculation		CT		Valve	V
	Crack	Circumferential	CC		Hydrant	FH
		Axial/Longitudinal	CL	Misc. Features	Diameter Change	DC
		Multiple	CM		Material Change	MC
	Fracture	Circumferential	FC		Spot Repair	SR
		Longitudinal	FL		Unknown Object	UO
		Multiple	FM			

Note: The information provided in this table is subject to change pending review of additional information and feedback from the industry experts and project stakeholders.

defects for coated and lined water mains. Following on the Water Research Centre (WRc) Manual of Sewer Condition Classification WRc (1993), codes are being developed for the following categories: (1) Structural Flaws/Defects; (2) Operational Flaws/Defects; (3) Construction Features; (4) Miscellaneous Features; and (5) Post-failure.

It is to note that a flaw or anomaly is defined as an imperfection that do not exceed acceptable limits, whereas a defect is defined as a flaw that does not meet a specific acceptance criteria (API, 2005; Brockhaus et al., 2014). According to these definitions, some large anomalies may not be classified as defects until they exceed acceptable limits according to some specified standard. To determine defect severity and to compute condition grades, fitness-for-service assessment, defined as "the quantitative analysis of the adequacy of component to perform its function in the presence of a defect" (Antaki, 2003), is required

Table 2 provides an example of selected anomalies and defects. Additionally, there are operational or service issues, such as coloured water (CW), low pressure (LP), taste (TST), odour/smell (SM), turbidity (T), reduced capacity (RC), and biofilm growth (BF). Deformation flaws and defects, such as dent (DD), ovality (DO), wrinkle or ripples (DW), and buckling (DB) may be classified into service or structural categories depending on their severity. It is possible that water mains made of different pipe materials have similar features, anomalies and defects. Therefore, in such instances, same codes can be used with a letter modifier indicating pipe material. However, there are specific features, anomalies and defects that are pertinent to one type of pipe material but irrelevant for other pipe materials. For such cases separate, material-related codes are needed. Further work to reach industry consensus on flaws and defects categories, codes, description, and measurement is in progress.

Determine Defect Severity and Compute Condition Grades

It is to note that a *flaw* or *anomaly* is defined as an imperfection that do not exceed acceptable limits, whereas a *defect* is defined as a flaw that does not meet a specific acceptance criteria (API, 2005; Brockhaus et al., 2014). According to these definitions, some large anomalies may not be classified as defects until they exceed acceptable limits according to some specified standard. To determine defect severity and to compute condition grades, fitness-for-service assessment, defined as "the quantitative analysis of the adequacy of component to perform its function in the presence of a defect" (Antaki, 2003), is required.

The condition grading and rating models can include simple screening tools to more sophisticated methods based on advanced statistical analyses, soft computing and machine learning techniques, and mechanistic models. The Water Research Foundation sponsored a number of initiatives to investigate the long-term behaviour of metallic (ductile iron, cast iron, and steel), plastic (PVC and PE) and asbestos cement water mains (see for example, Burn et al. (2005); Davis et al. (2007); Hu et al. (2010); Makar et al. (2005); and Rajani et al. (2011)). The findings from these investigations need to be further explored and synthesized to develop a reliable condition grading system. It is to note that some existing studies on water mains condition rating do consider distress indicators. However, they are not frequently used in practice because majority of the municipalities do not carry out water main inspections on regular basis. Additionally, existing studies report a number of limitations and unanswered questions because of the lack of adequate data. For example, Makar et al. (2005) recommends to carry out field verification of their results and to perform additional work on failure analysis. Thus, the development of a defect scoring and condition grading scheme needs further assessment of various types of defects and their significance in pipelines failures.

Steps 5: Decision Making

The decision support process determines when to rehabilitate or replace a pipe or to follow-up with one of the four evaluation tiers at another time depending on defects' severity and pipe condition grade. This involves risk-based (likelihood and consequence of failure) approach used by many water utilities. Existing tools and models, as discussed in Halfawy et al. (2008) and Moglia et al. (2006), with inputs from Steps 2 through 4, can be used for determining renewal priority. The result provides three options for managing a pipe that include: (1) no renewal and re-assign the pipe to one of the four inspection tiers; (2) renewal by rehabilitation; and (3) renewal by replacement

Step 6: Rehabilitation and Replacement Decisions

This step determines how to rehabilitate or replace a pipe identified in Step 5. A GIS-based model that takes into account asset parameters and condition grades provides options for renewal of the pipeline. The renewal options include: (1) open cut replacement; (2) in- line replacement with or without upsizing; (3) structural or semi-structural spot repair; and (4) structural or semi-structural lining system.

CONCLUSIONS

For water distribution systems, there are specific standards and statutory and regulatory requirements to ensure and protect drinking water quality within distribution and transmission mains. However, unlike plants piping or oil and gas pipelines industries, there are no established standards or guidelines for quantitative assessment of water mains defects. This paper presented the framework for the development of a standard water main condition classification system. The project sponsored by the Water Research Foundation involves municipalities and water utilities, water main inspection technology companies, and industry experts from the UK, USA and Canada. Existing water main condition rating systems are based on inferential indicators and/or pipe breakage history data. They do not take into account the condition assessment data and pipe anomalies. The proposed framework defines three levels of data to cater for general paucity of water mains condition assessment data and incomplete asset inventories. A simple risk-based methodology is developed to prioritize and categorize pipes into four tiers of condition assessment.

Failure mechanisms and modes, and anomalies for various types of pipe materials are reviewed and preliminary codes for pipe features and defects are presented. There is no single technology that can detect all the flaws, defects and features in a water main. Therefore, some codes may not be applicable for a particular technology. Furthermore, depending on pipe material, different types of anomalies and features can be expected in a water main. The code and feature descriptions will inform the relevant pipe material(s) for which a particular code will be applicable. To improve and enhance coding accuracy and consistency, a training/certification program can be useful for inspection operators and data analysts.

Additional work is needed to develop a consistent approach to assess the significance of defects and to develop weighting criteria for various types of defects. For this purpose the information from other Water Research Initiatives that investigated long-term performance of metallic (cast iron, ductile iron, and steel), plastic (PVC and PE) and asbestos cement needs to be synthesized. Furthermore, best practice guidelines, standards and specifications from wastewater pipelines, industrial piping and petroleum pipelines condition assessment can be adapted and modified for water main condition classification and reporting purposes. Water main condition grades can then be used to formulate asset management strategy for water main distribution pipelines. A standard water main condition classification system will help municipalities, water utilities, consultants, contractors, regulatory agencies, and inspection technology vendors use common language to describe pipe defects and features. This will also improve data quality, enhance inspection accuracy, and provide consistent data for effective decision making and policy development for sustainability of water distribution systems.

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Capital Planning for Shawnee County, Kansas, the Easy Way

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Abstract

The Public Works Department of Shawnee County, Kansas recently completed the first project phase of a program to clean, televise and evaluate the pipes of their sanitary sewer system. The current project consisted of the cleaning and televising of approximately 105,000 ft of sewer pipes in 1,350 acres of four of the County's sewer districts. The pipes were inspected and defects were identified and rated using the National Association of Sewer Service Companies (NASSCO) pipeline assessment and certification program (PACP) scoring system. In addition, all of the pipe segments and their associated defect data were linked to the County's geographic information system (GIS). Bartlett & West developed computation algorithms to calculate overall rating scores for each pipe in all four sewer districts. The pipes in all districts were then ranked for their comparative condition. Based on the pipe score, each pipe was recommended for improvement consisting of either replacement, point repairs, or lining. The costs associated with each pipe improvement were also automatically calculated using current cost factors. The website has provisions for multiple and nested queries. However, a standard set of queries was developed to allow rapid evaluation and computation of repair and rehabilitation costs. These standard queries can be made with as little as three clicks of the computer mouse. This "easy button" capability allows for rapid collection system understanding by County decision makers for use in planning their capital improvement plan (CIP) and assessing the need for rate increases.

CAPITAL PLANNING FOR SHAWNEE COUNTY

The Public Works Department of Shawnee County, Kansas with the assistance of Bartlett & West, Inc. recently completed the first project phase of a program to clean, televise and evaluate the condition of the pipes of their sanitary sewer system. The purposes of the program are to collect the information necessary to begin the development of a CIP, and to support a user rate study to determine rate modifications necessary to fund the CIP.

In addition, the Public Works Department is in the midst of ongoing discussions with the City of Topeka, KS for the City to purchase the County collection system and take over all operation and maintenance activities. The compiled collection system condition information will provide the two entities the understandings to set a purchase cost that is agreeable to both parties.

The current project consisted of the cleaning and televising of approximately 105,000 ft of sewer pipes in 1,350 acres of four of the County's sewer districts. The four sewer districts were Sherwood Hills, and District Nos. 2, 6 and 65. The Sherwood Hills district made up the majority of the work with 1,070 acres that adjoin Sherwood Lake and are located southwest of the City of Topeka just outside the City limits. Figure 1 shows the general project location of Sherwood Hills to the southeast of the City of Topeka.

This paper focuses primarily on the results of inspections in the Sherwood Hills district. The pipes were inspected and defects were identified and rated using the National Association of Sewer Service Companies (NASSCO) pipeline assessment and certification program (PACP) scoring system. In addition, all of the pipe segments and their associated defect data were linked to the County's geographic information system (GIS).

Computation algorithms were developed to calculate overall rating scores for each pipe in all four sewer districts. The computations consider various pipe

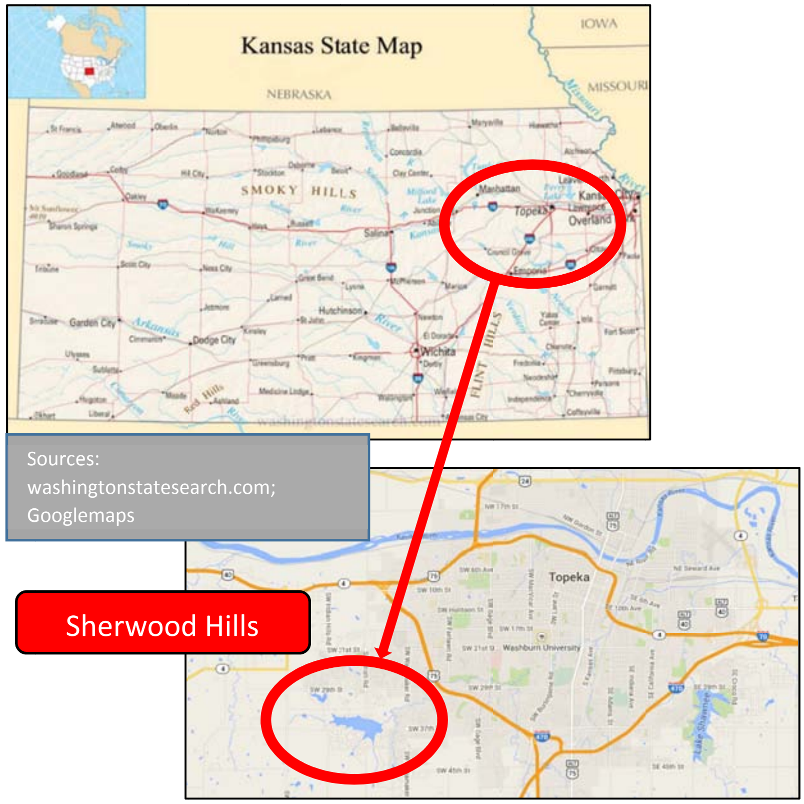
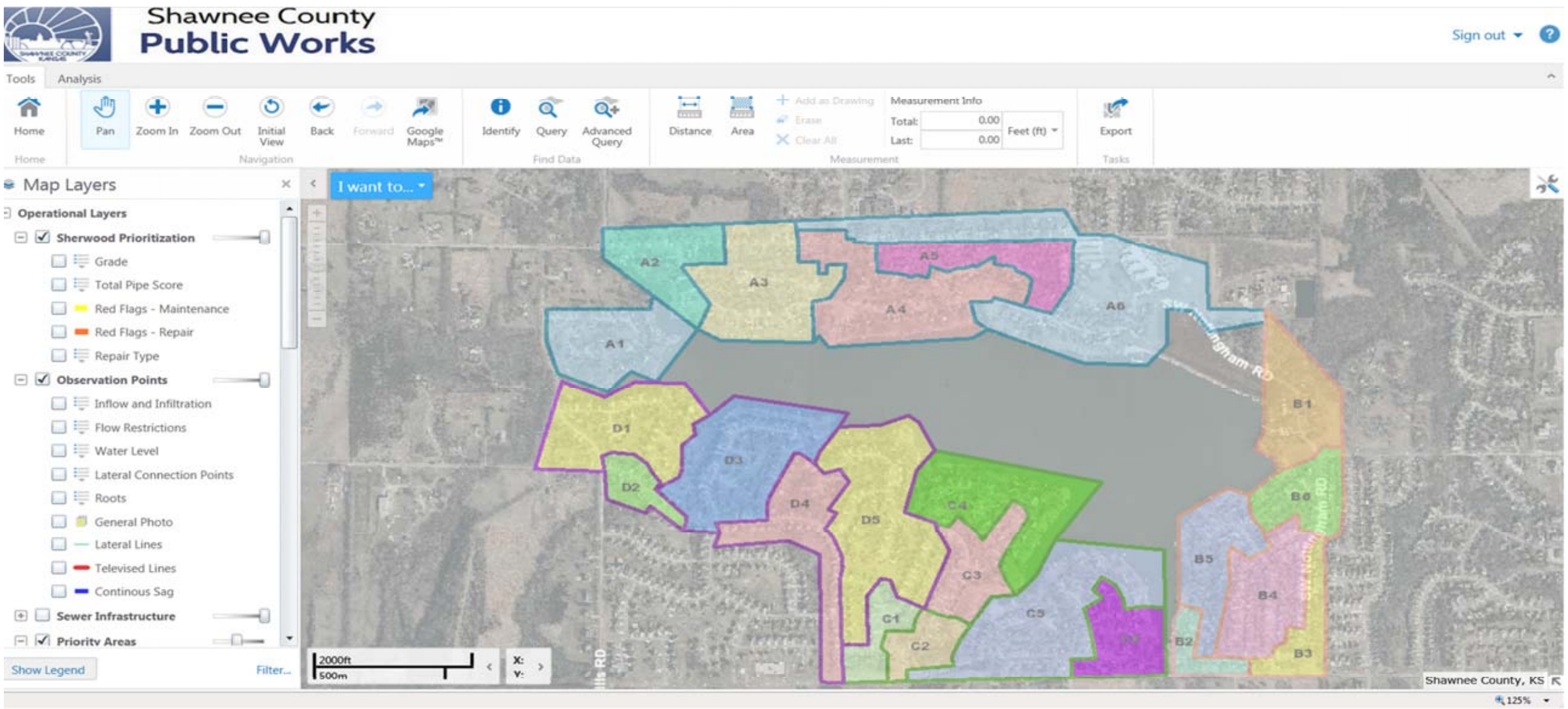


Figure 1 – Shawnee County Public Works, Sherwood

defects and observations that include: inflow and infiltration; flow restriction; water level; lateral connection points; roots; grease deposits; and sag points. The NASSCO rating for each defect was used with a selectable weighting factor, and an overall pipe score was computed. The pipes in all districts were then ranked for their comparative condition. In addition, based on the pipe score, each pipe was recommended for improvement consisting of either replacement, point repairs, or lining. The costs associated with each pipe improvement were also automatically calculated using current cost factors. All of the inspection, ranking and costing information is maintained on a website.

Figure 2 shows the home page of the Shawnee County Public Works website with the Sherwood Hills area in the main view area. The boundaries for the collection system sub-basins associated with the major basins A, B, C and D are delineated on the map. The left-hand area of the view screen indicates selectable map layers.

Figure 3 shows the total pipe scores computed for the pipes in the Sherwood Hills District. The figure indicates that the pipes in need of the greatest attention are in sub-basins C-5 and C-6, particularly C6. Planners and decision-makers can readily understand the situation at a glance. In discussion with Shawnee County staff, staff members stated that the map correlated exactly with their field experience and knowledge of the collection system. The C-5 and C-6 sub-basins are the oldest developed areas, and a predominantly clay pipe.



Multiple map layers are provided including conduit prioritization, observation points, sewer infrastructure (manholes, lift stations, sanitary sewer pipes, and force mains), sewersheds, and parcel boundaries

Figure 2 – Shawnee County Public Works Website Home Page

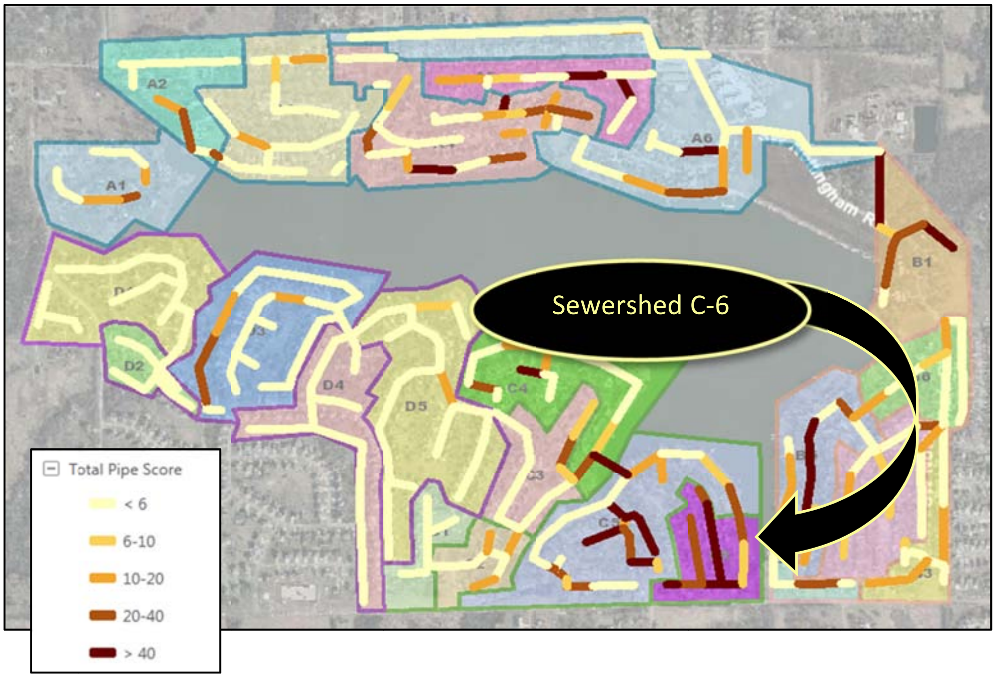


Figure 3 – Sherwood Hills Total Pipe Scores

Figure 4 shows another way of looking at the inspection results. The website provides layers in which pipes categorized as "red flag – repair" and "red flag – maintenance" may be highlighted on the area map. "Red flag – repair"

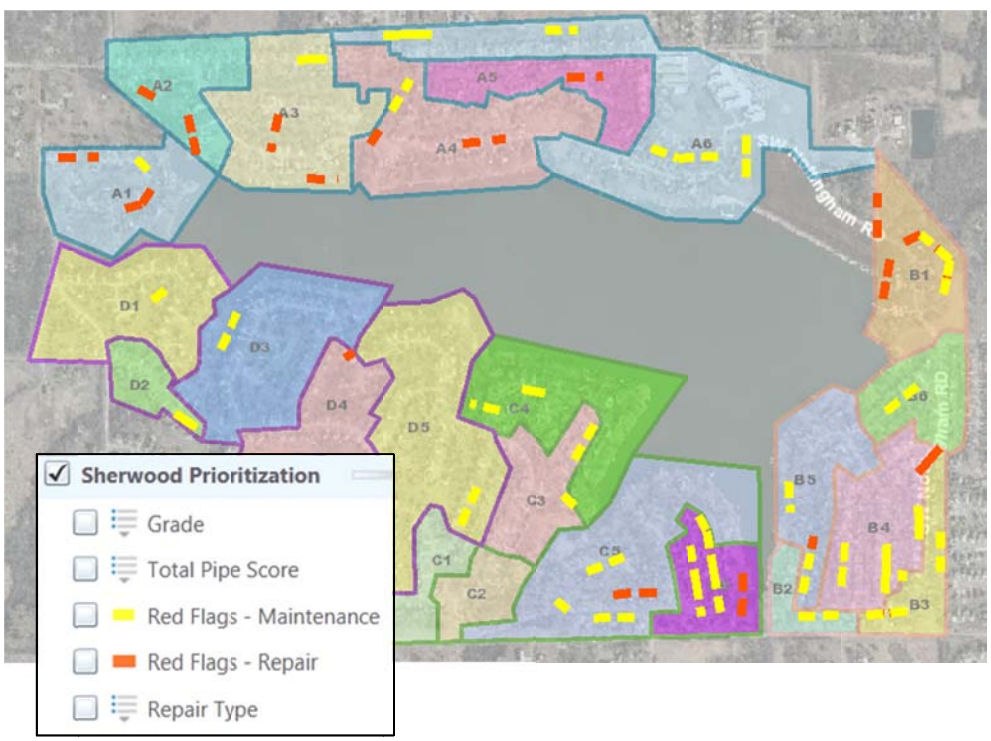


Figure 4 – Sherwood Hills, Red and Yellow Flag Pipes

indicates pipes that are in need of immediate replacement or extensive repair work to avoid serious consequences such as pipe collapses. "Red-flag – maintenance" indicates pipes that are in need of immediate maintenance such as heavy cleaning to avoid excessive system surcharging and basement backups.

Figure 5 shows the Sherwood Hills area with the layer for roots selected for viewing. Once again, in discussion with Shawnee County staff, the map correlated exactly with their field experience and knowledge of the collection system. Historically, the C-5 and C-6 sub-basins have been observed to have the greatest incidence of root-related maintenance problems. It is also interesting to note the correlation between the observed root incidences, and the location of the pipes red flagged for repair or maintenance in Sub-basin B-1, shows a high correlation between root incidences and red flag repair needs.

Figure 6 shows sub-basins C-5 and C-6 with observation layers selected for inflow and infiltration; flow restrictions; and roots with inflow and infiltration. Each observation type may be selected singly or in combination with as many other observation types as a user desires.

The website also provides functionality that allows users to identify observation types and directly link to associated inspection records in the form of tables, photographs, and videos. Figure 7 shows a root incidence photograph linked to a map with root incidences selected. Figure 8 shows the linkages between pop-up windows that allow a user to quickly call up tabular data and video inspections of the tagged observation.

In addition to its graphic capabilities, and quick linkages to data and videos, the website has provisions for multiple and "nested" queries. Multiple and nested queries are a powerful tool to perform in-depth analyses of the collection system data. These queries can be developed based on a user's needs or interests. However, the County wanted a capability that would allow the "casual" user of the website to readily consolidate the recommended pipe improvements into groups that could be considered packages that could be bid as projects.

A standardized query that became known as the "easy button" was developed for the website. The easy button allows a user to prepare reports with three clicks of the mouse. These reports automatically group pipes by district or defined sub-drainage areas to provide recommended individual projects.

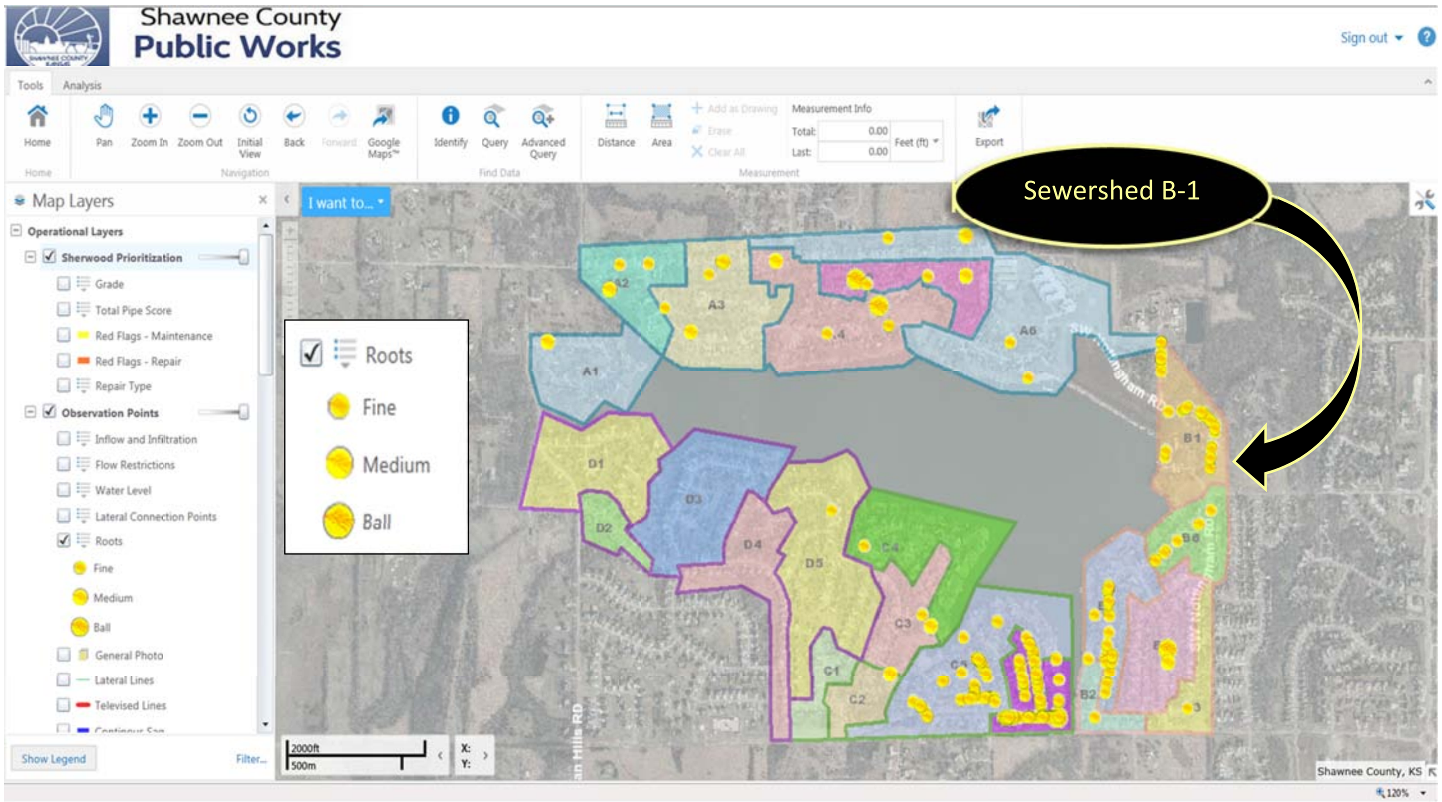


Figure 5 – Sherwood Hills, Root Incidences

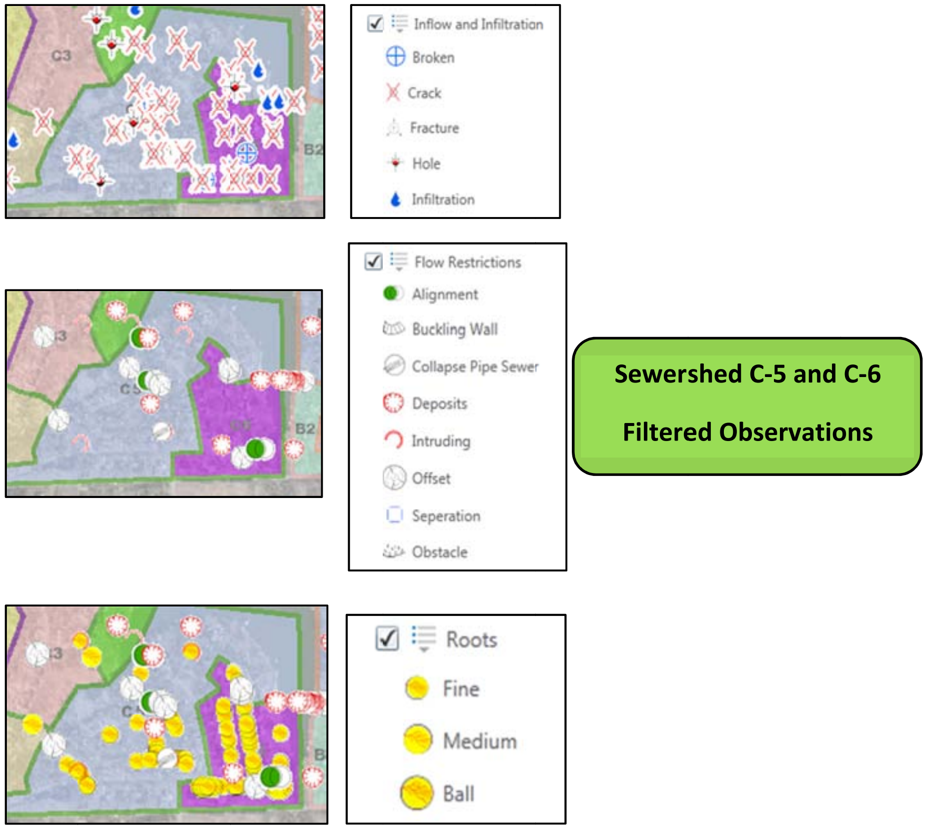


Figure 6 – Sherwood Hills, Filtered Observation Types

Figure 9 presents a report for sub-basin C using the "easy button." Backed by the computational algorithm, the query was made, and the report was generated in less than a minute. The report confirms the initial visually-based notion that sub-basin C-5 has the greatest amount of repair and maintenance work.

Similar reports can be prepared for the overall Sherwood Hills district, and for all of the sewer districts inspected. The County can then prioritize the projects as they see fit, and budget for the proposed work. The great advantage of the website is that as new data are collected and input, the prioritization of repair and maintenance needs are automatically and immediately updated. The County's CIP will always reflect up-to-the-minute information.

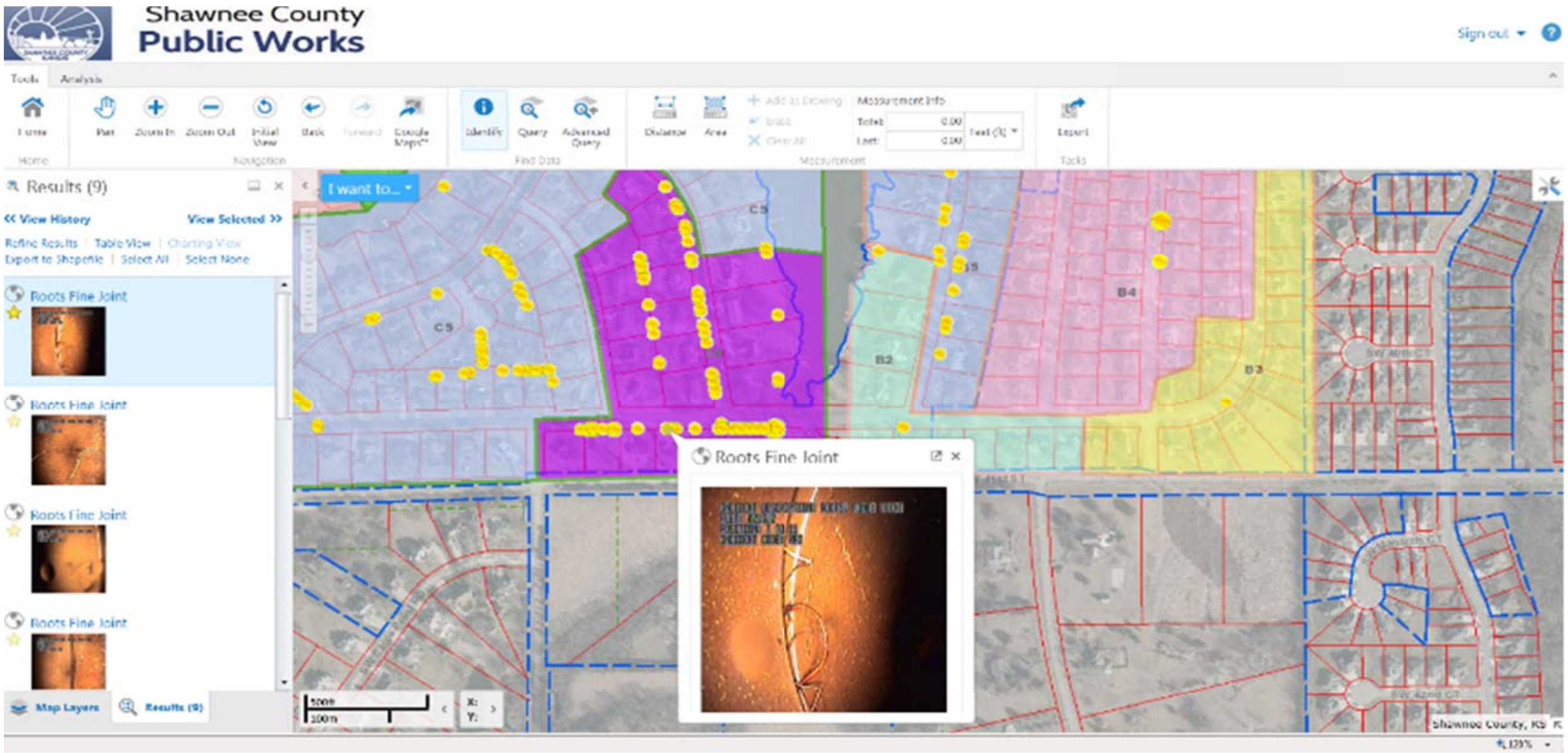


Figure 7 – Sherwood Hills, Photograph Linked to Observation Map

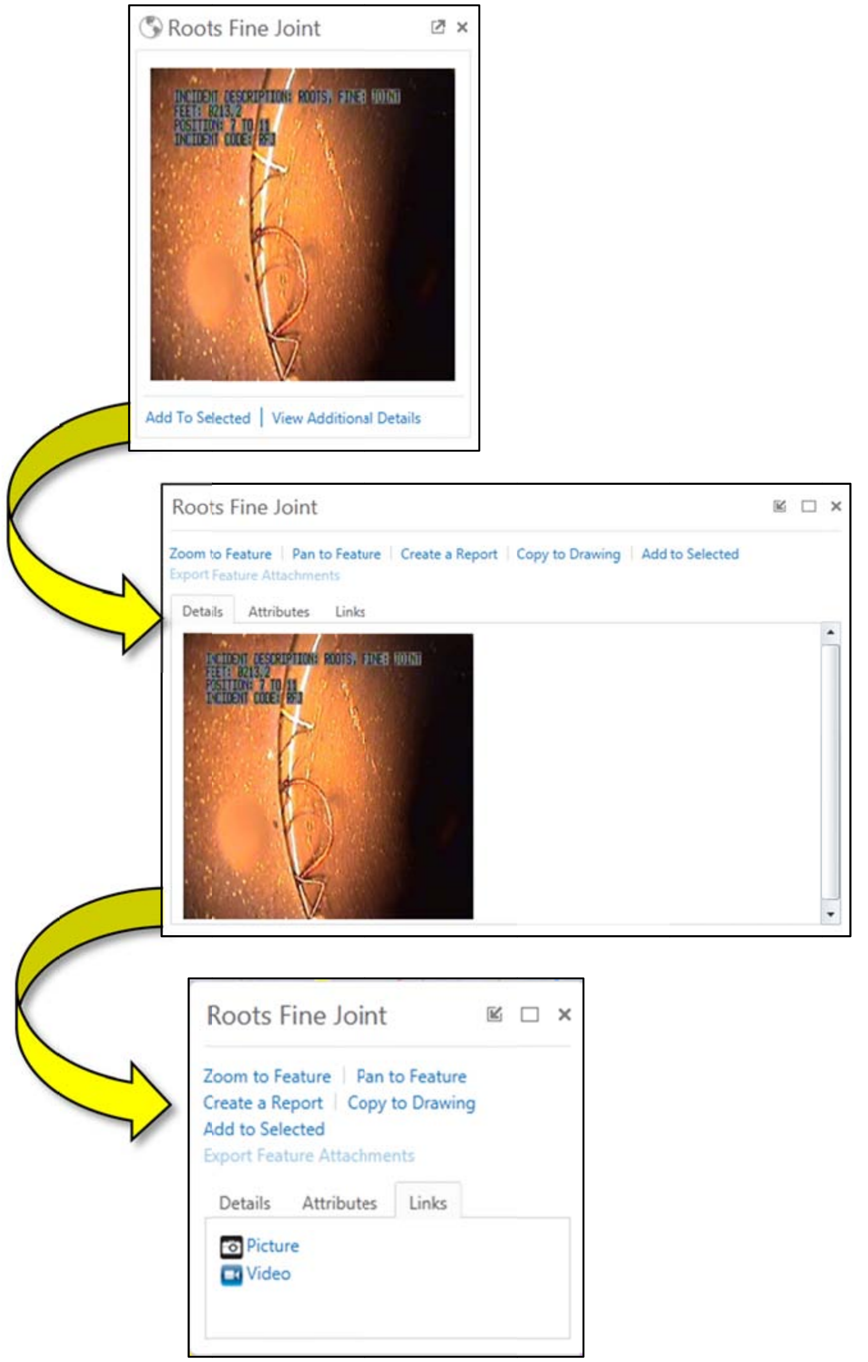
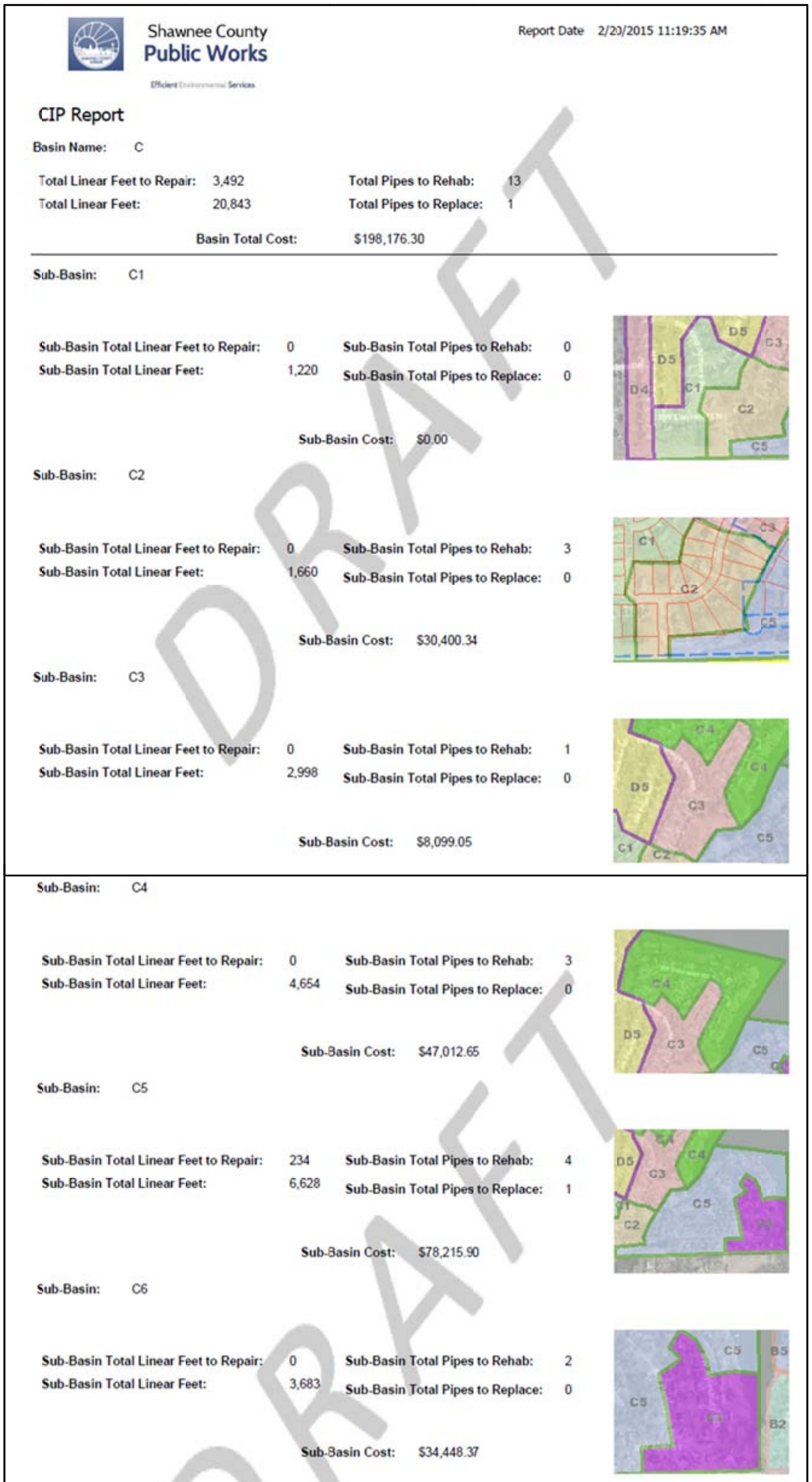


Figure 8 – Sherwood Hills, Information, Photograph and Video Links



Shawnee County
Public Works

Report Date 2/20/2015 11:19:35 AM

CIP Report

Basin Name: C

Total Linear Feet to Repair: 3,492
Total Linear Feet: 20,843
Total Pipes to Rehab: 13
Total Pipes to Replace: 1

Basin Total Cost: $198,176.30

Sub-Basin: C1

Sub-Basin Total Linear Feet to Repair: 0
Sub-Basin Total Linear Feet: 1,220
Sub-Basin Total Pipes to Rehab: 0
Sub-Basin Total Pipes to Replace: 0

Sub-Basin Cost: $0.00

Sub-Basin: C2

Sub-Basin Total Linear Feet to Repair: 0
Sub-Basin Total Linear Feet: 1,660
Sub-Basin Total Pipes to Rehab: 3
Sub-Basin Total Pipes to Replace: 0

Sub-Basin Cost: $30,400.34

Sub-Basin: C3

Sub-Basin Total Linear Feet to Repair: 0
Sub-Basin Total Linear Feet: 2,998
Sub-Basin Total Pipes to Rehab: 1
Sub-Basin Total Pipes to Replace: 0

Sub-Basin Cost: $8,099.05

Sub-Basin: C4

Sub-Basin Total Linear Feet to Repair: 0
Sub-Basin Total Linear Feet: 4,654
Sub-Basin Total Pipes to Rehab: 3
Sub-Basin Total Pipes to Replace: 0

Sub-Basin Cost: $47,012.65

Sub-Basin: C5

Sub-Basin Total Linear Feet to Repair: 234
Sub-Basin Total Linear Feet: 6,628
Sub-Basin Total Pipes to Rehab: 4
Sub-Basin Total Pipes to Replace: 1

Sub-Basin Cost: $78,215.90

Sub-Basin: C6

Sub-Basin Total Linear Feet to Repair: 0
Sub-Basin Total Linear Feet: 3,683
Sub-Basin Total Pipes to Rehab: 2
Sub-Basin Total Pipes to Replace: 0

Sub-Basin Cost: $34,448.37

DRAFT

Figure 9 – Sherwood Hills, Sub-Basin Repair Report

CONCLUSION

In discussions regarding the utility of the website, Shawnee County staff indicated that the mapping of pipe defects correlated exactly with their knowledge of the collection system and records for response to general complaints and incidences of basement backups. The website is considered by the County a very accurate tool to understand the status of the collection system.

The "easy button" is recognized by the County as a tremendously useful tool that promotes rapid understanding of the condition of their collection system and paves the way for accurate proactive planning.

Shawnee County plans to begin the next phase of work in the spring of 2015. This phase may include repair and rehabilitation of pipes identified in the first phase as being most in need of repair. As repair and rehabilitation work proceed, the estimated costs provided through the website will be evaluated for accuracy, and the cost estimating algorithms will be refined.

REFERENCES

National Association of Sewer Service Companies (NASSCO). (2010). *Pipeline Assessment and Certification Program, Version 6.0.1.*

Assessment of a Critical Raw Water Infrastructure for the City of San Diego

El Monte Pipeline Inspection and Condition Assessment Project

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Abstract

A team lead by ARCADIS, U.S. (ARCADIS) was retained by the City of San Diego (City) to determine the condition and remaining useful life of the City's 75-year old raw water transmission main, the El Monte Pipeline. As this pipeline ages, the City has taken a proactive approach to investigate this critical infrastructure in numerous phases. This is the first time the pipeline's condition has been evaluated and the assessment consists of various non-intrusive and intrusive assessment and inspection methods along its 12.2-mile long alignment. Assessment involves performing acoustical testing at 300-foot intervals along a 2-mile section of 48-inch diameter pipe, internal inspection on accessible reaches of the pipeline that varies from 48-inch, 68-inch, and 72-inch diameter. Assessment also includes external excavations at locations of concern determined from the acoustical and internal inspections, and limited core sampling where the pipeline is easily accessible.

EL MONTE PIPELINE HISTORY AND BACKGROUND

During WWII, San Diego was home to numerous military bases and defense industries. As the war effort expanded, a new raw water supply pipe was needed. The Federal Works Agency built part of the El Monte Pipeline and the rest was constructed by the City from 1942 to 1949. Because steel was in short supply during the war, the pipeline varied from 48-inch to 68-inch Reinforced Concrete Steel Cylinder (RCSC) pipe, 68-inch Reinforced Concrete Pipe (RCP), and a 72-inch cast-in-place reinforced concrete section through a tunnel. The cross sectional steel area within the pipe wall (steel cylinder and reinforcing steel) varies for each pipe size and type of pipe as well as internal pressure.

The City treats raw water at their Alvarado Water Treatment Plant, supplied by the 12.2 mile El Monte Pipeline, which delivers water from the El Capitan and San Vicente Reservoirs, and San Diego County Water Authority's (CWA) 1st Aqueduct.

The pipeline has a capacity of 135 million gallons per day (mgd) and a maximum operating pressure of approximately 118 pounds per square inch (psi). The maximum static pressure is approximately 160 psi occurring near the abandoned Lakeside Pump Station where San Vicente 1 & 2 Raw Water pipelines connect into the 48-inch El Monte Pipeline just upstream of a flow control valve structure.

When the El Monte pipeline was constructed, the area was largely rural with minimal development, but is now vastly urbanized. Presently, the pipeline is located within public rights-of-way and easements as it runs through residential areas, commercial zones, Caltrans right of way, San Diego Metropolitan Transit System (SD MTS) Trolley right of way, and numerous cities and jurisdictional agencies within east San Diego County.

The condition assessment activities were broken into four sections along the pipe, generally corresponding to changes in diameter and materials of construction. Figure 1 shows the 12.2-mile El Monte Pipeline alignment with its corresponding sections.

Figure 1. El Monte Pipeline Alignment

Section I is located in the eastern unincorporated Lakeside area where it branches off the El Capitan Pipeline that connects to the El Capitan Reservoir. This portion of the pipeline is 2-miles long and runs westerly along major roads surrounded by residential areas and small businesses. Section I starts with a 48-inch plug valve followed by a venturi meter. Air valves and blowoffs exist at high and low points along the alignment. Only one surface access manway exists on the downstream end of this Section, with four below grade manways along the stretch.

Section II begins at the Lakeside Pump Station/El Monte Regulating Valve Vault where the 48-inch pipe connects to the San Vicente Pipelines from the San Vicente Dam where the pipeline increases to 68-inch diameter RCSC. Section II is approximately 6-miles long and runs through the unincorporated Lakeside area and numerous cities including Santee and El Cajon. The pipeline crosses under Gillespie Airport, a public airport once used by the US Army during WWII, a Caltrans Freeway and Freeway interchange flyover bridges, and Trolley tracks. The alignment crosses small businesses and residential areas before entering Grossmont Tunnel. Portions of this pipeline section have slopes up to 20 percent. The only isolation valves are located at the El Monte Regulating Valve Vault with air valves located at high points and blowoffs at low points. Eleven surface access manways are located along this portion of the alignment in addition to four below grade manways.

Section III is the Grossmont Tunnel where the pipe material changes to 72-inch cast-in-place reinforced concrete. This Section is approximately 1.2-miles long and cuts through a hill beneath a residential area located in the Cities of El Cajon and La Mesa. No appurtenances are located on this portion of the pipeline except an air vent at the high point. Access points to the Tunnel include the air vent and a Section II manway.

Section IV is approximately 3-miles long and connects the Grossmont Tunnel to the Alvarado Treatment Plant in La Mesa. Section IV pipe material changes to 68-inch RCP, and the alignment traverses through residential and business areas, crosses under Caltrans Freeway bridges, and through Trolley right of way. The only isolation valve is located in the Alvarado Water Treatment Plant. Air valves and blowoffs are located at the high and low points, respectively, and all manways are surface accessible.

INSPECTION AND CONDITION ASSESSMENT

The project scope was divided into three Phases: 1) records research, site visits, and technology recommendations, 2) work plan development based on Phase 1 recommendations, and 3) performing the physical inspections and condition assessment based on the Phase II work plan. This phased approach was used to facilitate scope flexibility in the event of unforeseen challenges, better manage costs, and provide input from City staff as the final scope was developed for each work phase.

Phase 1 consisted of reviewing existing records, assessing constraints, reviewing and recommending investigation technologies, and developing an assessment approach. Several technologies were assessed including electromagnetic technologies, inertial navigation, transient pressure monitoring, acoustic leak detection, and acoustic pipe wall evaluation. A majority of the technologies were eliminated due to minimal or no experience with the RCSC or RCP pipe, requirement of the pipeline to be drained, capability to only provide location data, and inability to work with pipe configurations such as open air vents. Acoustic leak detection and acoustical pipe wall evaluations were determined to be the more appropriate technologies for the given pipe geometry, and allow the use of one device to obtain two sets of data.

Based on the Phase 1 decisions, a work plan was developed under Phase 2 that provided detailed locations for inspection sites within the constraints of environmental clearances and permitting requirements. A Phase 3 schedule and cost estimate for the field inspections, subsequent engineering analysis and reporting were also developed as a part of Phase 2.

In Phase 3, a pipeline alignment survey was conducted using as-constructed records and physical site inspections to establish accurate pipe and appurtenance locations; and accurately locate pipe and appurtenances in easements, neighboring residential and commercial areas and other jurisdictional rights of way. The survey data also assisted in determining excavation sites for non-destructive testing. The alignment survey found that several appurtenances and actual pipeline alignments were not as shown on the as-constructed records. Some air valves, manways, and blowoffs were located at varying distances up to 70 feet from their record locations. These discrepancies made it difficult to accurately locate and excavate below grade manways for access points.

Acoustic testing was used to determine if leaks exist and determine the remaining wall stiffness while the pipeline remained in operation. If successful, in the future it would provide a means to assess pipe condition without requiring the pipe to be out of service. To perform the testing, two hydrophone sensors were mounted to the pipe exterior, air valves, blowoffs, or other appurtenances at approximately 250 to 500 feet apart at each cross sectional steel area within the pipe wall (steel cylinder and reinforcing steel) for each pipe size and type of pipe for wall stiffness testing and 1,000 to 5,000 feet apart for leak testing. Where no surface features were accessible, excavations to the crown of the pipe were made at equal intervals to provide the most accurate and consistent results.

To determine if leaks were present, a correlator listened for noise created by leaks and the location determined by the amount of time it takes the noise created to reach the sensor. To determine wall stiffness, a noise signal was introduced to the pipe outside of the test segment and the resulting sound wave velocity was measured from the generated sound wave as it traveled down the pipeline. Changes in wave celerity provided an indirect assessment of the average wall thickness. Wall stiffness tests were performed on pipe lengths having the same cross sectional steel area (steel cylinder and reinforcing steel) for each pipe size and type of pipe. The leak test results can accurately locate leaks, but the wall stiffness results were averaged over the distance between the sensors (usually 300-foot intervals). This average stiffness provided a general indication of possible pipe deterioration rather than identifying unique problem areas. Echologics was selected to perform non-destructive, acoustic testing for leak detection and to determine structural integrity/wall stiffness.

An internal visual inspection of the pipe was included as part of the Phase 3 assessment to locate and identify any problems areas, inspect areas of mortar lining loss or delamination, look for signs of corrosion, and pipe joint displacement. The pipe was also sounded to determine potential areas of lining or coating delamination, or bedding void space around the pipe. The internal inspection also provided a means to corroborate acoustical testing results.

ARCADIS' in-house inspection team performed the internal inspection and sounding on the pipe. We are currently preparing for external pipe inspections. To determine if intrusive inspections should be eliminated or reduced in the future, acoustical testing, internal inspection, and external inspection results will be compared to determine if a trend exists.

Echologics Testing

As previously noted, typically, sensor intervals were located every 300-feet for each different area of steel. Where drawings indicated a change in steel area, the sensors were placed 20-feet from the edge of pipe stick according to the as-constructed records to allow for some variation in the alignment. Each access point for the sensors was uncovered using vacuum excavation, with a 6-inch PVC casing installed to the top of pipe. Casings were filled with gravel and covered with asphalt after testing was complete to provide future testing access if desired. Section I testing was scheduled first as it was the shortest Section and the results could be verified with the internal inspection. Excavation and traffic control permits were obtained from the County of San Diego to perform this work.

Internal Inspection

Internal inspections required substantial preparation and coordination with City operations staff. Dewatering locations, ingress/egress locations, ventilation locations, and pump out flow rates were determined prior to entry and revised in the field as necessary.

This pipeline carries raw water from two reservoirs that are prevalent with Quagga mussels. Quagga mussels are an invasive species that have been introduced to many reservoirs and waterways in the area and any dewatering activities require the use of filters by the California Department of Fish and Wildlife to prevent further introduction to waterways. Because this is a raw water line, it does not carry any disinfection residual, so dechlorination was not required prior to discharge.

From a personnel safety standpoint it is desirable to have as many ingress/egress points as possible in case the team needed to exit the pipeline unexpectedly. Two of the Sections had inaccessible surface manways which made ingress and egress a concern if an issue developed as the team would have to perform a "there-and-back" inspection which requires more time and less flexibility when entering and exiting the pipeline. The buried manways were not as shown on the as-constructed records, so "there-and-back" methods had to be performed to locate the below grade manways using a sonde from within the pipe. That location would then be excavated for use the following inspection day. If blowoffs could not be located or unforeseen conditions existed, "there-and-back" inspections occurred as well. Where steep slopes were present, the team would enter from a high point and traverse to the low point. These "there-and-back" stretches were a concern for air flow through the pipe as there was not typically an exit point to keep fresh air moving through the pipe to an exhaust location. Where possible, air valves were opened to permit air flow, but self-contained breathing apparatus (SCBA) devices were carried by each crew member during each inspection in case low oxygen levels became an issue.

The internal inspection effort was broken into three separate shutdowns to facilitate the pipe being removed from service, dewatered and recharged, and due to the physical constraints of inspecting 12-miles of pipe. Also the inspection team had to accomplish all work within a maximum shutdown period of ten days during peak water demand months, as required by the City. Extended shutdowns can be requested during non-peak water demand months that can be a few days longer than the ten-day shutdown.

The City provided shutdown windows months in advance, and the inspection schedule was coordinated based on Section accessibility. Section III and IV internal inspections were scheduled first as all pipeline entry was provided through surface accessible manways. Section I was originally scheduled in mid-Fall to permit dewatering without Quagga filters as the El Capitan Reservoir is considered to have anoxic zones in warmer months that would prevent the Quagga mussels from surviving. That exception was removed by the California Department of Fish and Wildlife as Quagga mussels may be able to survive the anoxic conditions making Quagga filters mandatory year round and resulted in additional costs that delayed the inspection one and a half months. Section II was scheduled during non-peak water demand months as it is the longest Section and required an extended shutdown of an extra two days. Inspection data were collected using numerous devices including a camera for pictures, a computer tablet to take notes for corresponding pictures, and GoPro's for video. The tablets and pictures are recorded in Assethound, a mobile-based data collection and inventory software developed internally by ARCADIS. The software supports customizable tables to enter information that is uploaded to the Cloud at the end of each day and provides flexibility for the data to be accessed remotely from any location.

In order to perform the internal inspections, numerous permits had to be acquired for each jurisdictional agency for dewatering to storm drains and to gain access to easement areas as shown in Table 1.

Table 1. Internal Inspection Permits

Agency	Permit Required
County of San Diego	Traffic/Encroachment Permit
City of Santee	Encroachment Permit
City of La Mesa	Traffic/Encroachment Permit
Caltrans	Encroachment Permit
SD MTS	Durable Right of Entry Permit
Gillespie Airport	Airport Use Permit

External Inspection

Once acoustic testing and internal inspections were completed, the team evaluated potential sites to conduct external inspections and soils testing. This work is now beginning and locations will be selected based on areas where acoustic testing indicated a loss of wall stiffness, and areas where internal hollow soundings were noted or where visual inspection found cracked, damaged or missing mortar liner. Permits will be obtained from the appropriate jurisdictional agencies when the external excavation locations are determined.

When all testing, internal and external inspections have been completed, the team may recommend taking select core samples to further corroborate findings from the Echologics work and internal/external inspections. Four core samples remain within the project Scope but may not occur, as coring, even with appropriate repair, would affect the pipe integrity.

RESULTS AND RECOMMENDATIONS

Acoustical Testing

Acoustical testing was performed on Section I in August 2014 by Echologics, with their field work completed over a four day period. The leak detection testing determined that no leaks were present within the 48-inch pipeline. Pipe wall stiffness was calculated based on the available design information for each pipe segment's area of steel, liner thickness, and coating thickness making up each composite pipe cross section. Echologics compared acoustic readings taken over each 300-foot test segment to the segment found to have the highest wall stiffness in the Section. This comparison was used to determine how much theoretical wall loss occurred in each test segment regardless of the area of steel, so different areas of steel were compared with one another. The first 1,500-feet and last 600-feet of 48-inch were determined to have some of the lowest stiffness values compared to the test segment with the highest stiffness which was located about 1.5-miles from the beginning of the Section. Based on the as-constructed records, the first 1,500-feet of pipe in Section I was constructed with a lower area of steel than other test segments and could account for the lower stiffness values. According to the as-constructed records, the last 600-feet of pipe should have the most area of steel in the Section.

ARCADIS used the stiffness values provided by Echologics to perform a similar evaluation on the pipeline. Instead of determining the theoretical wall loss by comparing each test segment to that with the highest stiffness, the stiffness from each test segment with the same area of steel were compared to one another. This type of evaluation was performed because a test segment that originally had a lower area of steel than another segment is expected to have a lower stiffness. This method was used to determine if there was a trend between the internal inspection defects and the acoustic data.

Internal Inspection

The El Monte Pipeline underwent three separate shutdowns that included dewatering and inspecting the interior of each Section where accessible. The only areas not inspected were those where a blowoff could not be located to adequately drain the pipe, or an unknown dip in the pipe occurred resulting in a pipe full of water. This resulted in 97 percent of the pipeline being inspected over three shutdowns. Dewatering operations included lock out tag out by the City, contractor, and ARCADIS. Each blowoff used for dewatering had suction piping connected from the blowoff to a pump and a Quagga filter which had discharge piping routed to a flood channel, storm drain, or natural ditch as shown in Photo 1.

Photo 1. Typical Dewatering Operation

At each shutdown, estimated time for dewatering was calculated to determine contractor time and material necessary for the dewatering process. However, on Sections II, III, and IV, dewatering times were much longer than calculated. Leaking valves at the Treatment Plant and infiltration through Tunnel joints were the sources of added dewatering volume.

Section I inspection was performed during two different shutdowns in December 2014 and January 2015. The first shutdown consisted of entering the pipe through a surface accessible manway and performing a "there-and-back" inspection since none of the buried manways were located by surface excavation. One mile of the Section was inspected in December. After the December inspection, the team felt more confident in locating the buried manways from above grade, so inspection of the second half of Section 1 was added to the original scheduled January inspection.

As the Section I pipe was 48-inch diameter, it was not possible to do a walking inspection and crawling would be too physically demanding, take more time, and the full inspection distance would be difficult to achieve in the shutdown timeframe. Due to this, Section I inspections used pipe carts. The team was able to lie on their backs and push their way through the pipe and used their feet for braking on down slopes. Additional carts were used to carry SCBA equipment, data collection devices, and accessories.

During the December inspection, the pipe was entered through the surface accessible manway at the downstream end of the 48-inch pipe and the team inspected two-thirds of a mile of the pipeline. As no other surface accessible manways were available on the 48-inch, the inspection team used a sonde so as they internally located a buried manway, its surface location could be determined. This allowed a manway to be excavated to provide access the following inspection day and permitted an additional half mile of inspection. During the January shutdown, the team was able to excavate and locate two buried manways from above ground to permit entry into a portion of the pipeline that was isolated from the first inspection.

Overall, the interior of the 48-inch was in good condition except for the upstream and downstream ends. At the upstream end the pipe is smaller in diameter and numerous reducers exist on the venturi meter. Due to the number of reducers and venturi meter, the material could be cement mortar lined and coated steel or specials. The lining on the reducers is missing and a portion of the steel cylinder is separating from the concrete coating as shown in Photo 2. The 48-inch pipe was isolated from the system for a few months prior to the inspection causing the water in the pipeline to become anoxic. Due to this, the Quagga mussels detached from the circumference of the pipeline, where caught in the filters, and very few mussels were found in the pipeline.

Photo 2. Exposed Steel Cylinder at Joint

Section II inspection was performed in January 2015 and being a larger diameter, the means of transport was by bicycles taken apart and passed through surface accessible manways, then reassembled inside the pipe. The 68-inch pipe is large enough to walk through, but use of bicycles greatly reduced the inspection time to three days, where six days were expected for the 6-mile long Section, and it was less physically demanding for the team. Lights and baskets were mounted to the front of the bicycles which allowed storage of data collection devices and accessories as shown in Photo 3. SCBA equipment was worn on their backs.

Photo 3. Bicycle Inspection Configuration

The 68-inch RCSC was found to be in generally good condition except at the El Monte Valve Vault where there are numerous pipe material and diameter changes and the interior lining is nonexistent. A crack was found under the 67/52 Freeways interchange flyover bridges that were constructed in 2011. Large quantities of Quagga mussels were found on the bottom of the pipe downstream of the El Monte Valve Vault as shown in Photo 4.

Photo 4. Quagga Mussels

Section III inspection was performed in one day on bicycles in September 2014 and had the most defects of any Section. Leaks were found in the Grossmont Tunnel at more than half of the joints and were mainly located at or just above the pipe spring line as shown in Photo 5. Each leak was assigned a qualitative value of less than 1 gallon per minute (gpm) to 5 gpm based on photo observation. The joint leaks could result from a perched groundwater source or raw water leaking out from the Tunnel when full, and then returning into the tunnel as a result of dewatering. The Tunnel is the high point of the transmission main and watermarks were generally found at the spring line, so the Tunnel does not typically flow full. Quagga mussels were found attached to the circumference of the pipe along the Tunnel's entire length.

Photo 5. Grossmont Tunnel Joint Leaks

Section IV inspection was performed in two days in September 2014 at the same time the Tunnel inspection occurred and transport was by bicycle through surface accessible manways. The 68-inch RCP was in good condition with some crazing and mineral deposits on the coating and a few spalling locations. Quagga mussels were attached around the circumference of the pipe along the entire 3-mile length as shown in Photo 6.

Photo 6. Quagga mussels attached to RCP

External Excavations

To date, one external excavation has been performed based on an area of concern identified by the Echologics data. The analysis of the excavation has not been completed and will be included in the final report and results presented at the conference.

Core Sampling

One core sample was taken on a piece of 68-inch RCSC pipe that was removed in April 2014 during construction of flood channel improvements. The sample has been sent to a laboratory for analysis, but results have not been completed at this time.

Next Steps

With approximately six months remaining in the project schedule, the remaining assessment/inspection work consists of seven or more external excavations at locations recommended from the internal and acoustic inspections. Areas to be chosen for external excavation and inspection include long spans of pipe that when tapped resulted in hollow sounds during the internal inspection, areas of reduced stiffness noted in the acoustical testing, and where cracks or pulled joints were found. External inspection may also add data to determine why some areas of internal inspection versus acoustic analysis yielded widely divergent results.

ARCADIS compared the internal inspection defect findings with the pipe wall loss values for similar areas of steel to determine if any trends exist and whether one technology could be eliminated for future assessments. The defects found from the internal inspection did not always correspond to the results provided by Echologics and a trend could not be determined. For example, at some locations lower wall stiffness corresponded well to an area where a defect or long hollow sounds were found, but in other locations the results were widely divergent. In some cases Echologics showed decreased wall stiffness, but no defects were found during the internal inspection, and vice-versa. Reasons for the discrepancies could be a result of the acoustic testing averaging it's finding over the span of the test, or because the joints are not welded preventing the acoustics from passing through the pipe wall as intended. Because no trend could be determined, Echologics work was discontinued after Section I.

Over the next six months, the final report will be developed that will provide detailed results on the existing structural condition, remaining service life, and provide recommendations for future inspection frequency, assessment/inspection type/technology, and rehabilitation.

Preliminary recommendations for repair and rehabilitation to the pipeline include replacing a badly leaking butterfly valve located at the Treatment Plant to permit future inspections and minimize dewatering. Repairs to the Grossmont Tunnel which could include relining or sealing the cracks, holes and voids are also recommended. Rehabilitation or replacement of pipe at the upstream end of the 48-inch near the venturi meter and downstream end of the 48-inch near the El Monte Regulation Valve Vault are recommended. Rehabilitation or replacement near the El Monte Regulating Valve Vault where numerous pipe material and diameter changes exist are also recommended as the lining was no longer present. Pipe within the Alvarado Treatment Plant up to the valve should also be repaired or replaced.

Evaluation of Acoustic Wave Based PCCP Stiffness Testing Results

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Abstract

Prestressed concrete cylinder pipeline (PCCP) condition assessment methods include an acoustic wave propagation testing technology (ePulse) to identify sections of pipeline with reduced structural stiffness. A recent innovation, ePulse, is a survey level test that uses acoustic sensors to measure the velocity of an acoustic wave in a pipe segment. The wave speed in the pipe is a function of the wave speed in an infinite body of fluid and the pipe stiffness; therefore, changes in wave speed may indicate changes in the pipe wall stiffness along the test segment. Reduced pipe wall stiffness may be an indication of broken prestressing wires, lower prestress, deteriorated mortar coating, cracked concrete core and others. This paper presents a brief description of acoustic wave based pipe stiffness testing and the structural and failure risk analyses and external pipe inspection to evaluate the ePulse predicted pipe stiffness in several pipelines. Results of a case study including pipe stiffness testing on several sewer forcemains are evaluated based on variability of the measured pipe stiffness within a given pipe class, and comparison of the measured and nominal calculated pipe stiffness. We discuss parameters that may affect the results such as entrapped air, pipeline design properties and layout, appurtenances and other features. We discuss observations from the external inspection of excavated pipe to evaluate the results of ePulse predictions. Results of field inspections indicate correlation of observed distress with predicted reduced pipe stiffness and show that the use of this technology, coupled with structural analysis of the pipeline properties, can identify sections of pipeline with structural deterioration for further investigation.

METHOD OF APPROACH

Many utilities are adopting programs of pipeline asset management aimed to maintain the pipeline risk of failure at an acceptable level. Such programs are discussed in the Water Research Foundation "Best Practices Manual, Prestressed Concrete Cylinder Pipe

Condition Assessment – What Works? What Doesn't? What's Next?" (Zarghamee et al. 2012[1]) and generally include periodic condition assessment, failure margin analysis, identification of pipe pieces with unacceptable failure risk, and repair or replacement of such pipes. Selection of sections of pipelines for condition assessment and the technology used for condition assessment should be based on a desktop study to determine criticality, which accounts for the pipeline likelihood of failure, consequences of failure, and system constraints. This approach is similar to the tiered (pyramid) approach to condition assessment adopted by some utilities. Each pipeline may be inspected with sufficient granularity to determine the appropriate course of action for renewal. Options would include total replacement, partial or full rehabilitation, or potentially no action at all. This approach mitigates the risk of failures while optimizing the use of precious capital.

OVERVIEW OF ePulse TECHNOLOGY

ePulse can be used as a screening tool to prioritize long lengths of PCCP sections based on the measured average structural stiffness over approximately 150 ft long segments of the pipeline. ePulse uses two surface mounted sensors, one located at each end of the test segment, to measure the velocity of an acoustic wave in the pipe generated by a source located outside of the test segment (not between the sensors) as shown in Figure 1. The wave speed in the pipe is a function of the wave speed in an infinite body of fluid and the pipe stiffness; therefore, changes in wave speed indicate changes in pipe wall stiffness along the test segment.

If an accurate measurement can be made of the acoustic wave velocity, it is possible to calculate the average stiffness of the pipe between the two sensors. For metallic and asbestos cement pipes, this stiffness is in turn used to calculate the remaining wall thickness. For PCCP the wave velocity is used to calculate an average composite pipe wall stiffness that may be affected by loss of prestress, wire breaks, and concrete degradation and cracking. This provides an indication of general loss of structural strength over the test interval, and provides a valuable screening tool to help determine which portions of a PCCP pipeline may be suffering from higher rates of degradation than others. For PCCP, test spacing must be no greater than about 10-15 pipe pieces, up to about 200 feet long.

The results of ePulse testing can be used as a relative measure of pipe condition in multiple segments of nominally similar pipe, or as a part of a pipeline screening and condition assessment program that includes pipe structural and failure risk analysis, and localized external inspections.

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[1] Zarghamee, M.S., R.P. Ojdrovic, and P.D. Nardini. 2012. "Best Practices Manual, Prestressed Concrete Cylinder Pipe Condition Assessment – What Works? What Doesn't? What's Next?" Denver, CO: Water Research Foundation.

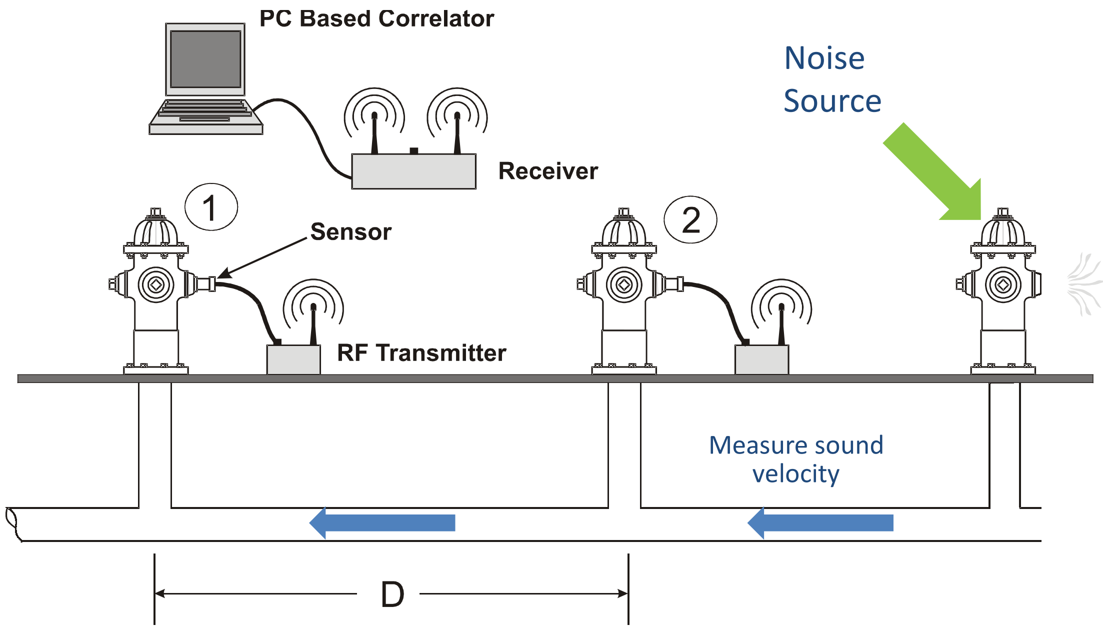

Figure 1. Schematic of ePulse testing configuration.

DESKTOP STUDY

Structural Evaluation

We perform structural evaluation of the pipeline for circumferential effects of the applied internal and external loads in accordance with AWWA C304 Standard – Design of Prestressed Concrete Cylinder Pipe to determine if the pipe designs meet the limit states for the design loads and for the currently applied loads. The current pipe design procedure uses a limit states approach in accordance with AWWA C304 based on meeting certain serviceability, damage, and strength limit states and is implemented in the computer program UDP (Unified Design Procedure). The loads applied to the PCCP consist of the maximum internal working and working-plus-transient pressures, pipe and fluid weights, earth load, live load, and prestressing force. The results of structural evaluation indicate if any pipe damages might be expected due to actual loads that may or may not be higher than original design loads.

Failure Risk Analysis. The failure risk of distressed pipes is determined based on the size of the prestress loss zone and the maximum pressure (working-plus-transient pressure). Pipe weight, fluid weight, and earth load on the pipe are also considered in the failure risk analysis. The results are summarized in the form of failure risk curves (FRCs) that show the maximum pressure in the pipe versus the effective number of broken wires (size of prestress loss zone) for different limit states (i.e., serviceability, damage, and strength). Each risk curve is developed for a specific pipe design and soil-cover height, predominant in each pipeline. The limit states quantify the level of damage in the pipe and are used to assign repair priorities.

The FRCs are developed from nonlinear finite element analyses (FEA), calibrated by hydrostatic pressure testing of PCCP, and validated by field inspection of various

PCCPs with broken wires at other pipelines (Ojdrovic et al. 2011[2]). The results of the FEA are used to determine the pressures and the number of broken wires (length of distress) that produce different levels of distress associated with the serviceability limit state (i.e., onset of core cracking), the damage limit state (i.e., structural cracking of core and increase in wire stress adjacent to the broken wire zone), and the strength limit state as a function of the length of distress (Zarghamee et al. 2003[3] and Erbay et al. 2007[4]).

For some pipelines, loss of prestress and corrosion and perforation of steel cylinder may lead to changes in pipe wall stiffness that can affect the wave propagation speed through the pipeline and be detected by ePulse. Based on the pipe design properties and the applied internal pressures, some pipelines will leak well prior to rupture as corrosion and perforation of the steel cylinder occur prior to reaching the pipe ultimate strength. FRCs can be used to identify appropriate inspection tools based on the PCCP sensitivity to wire breaks and likelihood of leakage before rupture, and evaluate detailed inspection results by relating the length of prestress loss to the pipe failure margin.

EVALUATION OF ePulse TEST RESULTS

We evaluated the results of ePulse testing in three projects involving PCCP. The first was a blind test involving a tear-down inspection of 20 in. diameter lined cylinder pipe (LCP) from a water pipeline, the second was a condition assessment of several sewer forcemains, and the third was a condition assessment of several water pipelines in and around levees. Condition assessment of the forcemains is discussed in this paper.

ePulse measures the velocity of a wave in a pipeline. We performed calculations investigating the sensitivity of the PCCP composite stiffness calculation to changes in the various parameters associated with wave propagation in pipes. The relationship between velocity of a wave in the fluid in a pipe, the stiffness of the elastic fluid, and the stiffness of the elastic pipe can be written as:

$$\frac{1}{\rho_w v^2} = \frac{1}{K_w} + \frac{2}{K_p}$$

where ρ_w is the density of the fluid, K_w is the bulk modulus of the fluid, and K_p is the measured hoop stiffness of the pipe. This relationship is non-linear. The pipe

[2] Ojdrovic, R.P, P.D. Nardini, and M.S. Zarghamee. 2011. "Verification of PCCP Failure Margin and Risk Curves," *Pipelines 2011: A Sound Conduit for Sharing Solutions*, Seattle, WA, 1413-1423.

[3] Zarghamee, M.S., D.W. Eggers, R.P. Ojdrovic, and B. Rose, "Risk Analysis of Prestressed Concrete Cylinder Pipe with Broken Wires," *Proceedings of ASCE Specialty Conference Pipelines 2003*, Baltimore, MD, 2003.

[4] Erbay O.O., M.S. Zarghamee, and R.P. Ojdrovic, "Failure Risk Analysis of Lined Cylinder Pipes with Broken Wires and Corroded Cylinder," ASCE Pipeline Conference, Boston, MA, 8 to 11 July 2007.

stiffness K_p calculated from measured velocity and assumed fluid density and bulk modulus can be very sensitive to small changes in the other quantities.

Uncertainties in either the velocity measured, or the assumed values for fluid density and bulk modulus may have a magnified effect in the calculated pipe stiffness. The velocity measurements are sensitive to accurate distance measurement between sensors. Over the range of temperatures expected in pipes the density of water changes little, but the bulk modulus may vary by 10% between 32°F and 68°F. Furthermore, if the pipelines convey sewage, the density and bulk modulus of sewage may be highly variable with time and between pipelines. Entrained air may have significant effects on wave speed. Pipeline layout and construction including fittings and special pieces and different pipe classes within a test segment may make interpretation of the results more complicated. The nominal pipe stiffness is based on the pipe design. Differences between the nominal and the measured pipe stiffness may be due to pipe distress, variations in material properties, changes in pipe manufacture (i.e., lower wire prestress), and/or other uncertainties, that may affect the measurements.

CASE STUDY - 16 IN. TO 42 IN. DIAMETER SEWER FORCEMAINS

Echologics utilized ePulse technology along seven sections of the sewer forcemain system. Each of the seven sections is subdivided into two or three test segments for a total of eighteen test segments. Testing locations were selected based on the internal operating pressures of the pipelines and results of preliminary failure risk analysis. Each pipeline segment was between 79 ft and 166 ft long and was located on a relatively straight portion of the pipeline. The ePulse results are an average composite stiffness over each test segment and provide relative stiffness values to compare sections of the same pipeline.

We selected four external inspection locations, one in each pipeline, by identifying test segments with low composite stiffness relative to other test segments of the same pipe class. Selection of specific inspection locations within each of these segments was based on accessibility.

The scope of the external inspection was as follows (inspection was performed while pipelines were in operation):

- Perform visual and sounding inspection of the exposed portion of the pipe and check for cracks, hollow-sounding areas, signs of leakage, signs of corrosion, and other distress. One pipe length was exposed down to below springline.

- Remove mortar coating to expose at least one prestressing wire and collect the removed mortar coating pieces for laboratory testing.

- Measure diameter and spacing (if possible) of prestressing wires and mortar coating thickness for comparison with input used in analysis.

- Measure half-cell potentials according to ASTM C876 on the exposed portion of the pipe surface.

- Remove mortar coating from areas with likely corrosion to expose the steel cylinder and prestressing wires for inspection.

The results of inspection show the following:

- We did not find any hollow-sounding areas or cracks in the exposed portion of the four pipes.

- Pipeline A (24 in. diameter): Based on half-cell potential measurements, we made four openings in the coating and observed that the mortar coating had small patches that were not well bonded to the steel cylinder (Figure 2). We observed corrosion on the prestressing wires and steel cylinder and one broken prestressing wire, and a small hollow sounding section of the cylinder beneath the broken wire (Figure 3). One wire on each side of the broken wire moved side-to-side when tapped with a hammer, indicating that the wire may have been broken under the mortar coating away from the window.

- Pipeline F (33 in. diameter): The mortar appeared to be well-bonded but it was possible to remove it in large pieces. We also observed a large depression in the pipe's mortar coating that measured approximately 0.6 in. in depth.

- Pipeline M (42 in. diameter): The mortar appeared to be well-bonded and hard; no significant corrosion of steel cylinder and wires.

- Pipeline W (16 in. diameter): The mortar appeared to be well-bonded, but it was possible to remove it in large pieces. The steel cylinder and prestressing wires showed some pitting on the surface of the prestressing wires and some section loss of the steel cylinder.

Figure 2 - Corrosion product and poor bond and steel cylinder surface of Pipeline A.

Figure 3 - Brittle broken prestressing wire and 2 in. by 2 in. hollow sounding area of steel cylinder behind wire break in Pipeline A.

Laboratory Testing

We performed laboratory testing to determine the corrosivity of soil near the pipeline and to determine the quality and condition of the mortar coating.

The soil corrosivity test results indicated soils with low soil resistivity in five of the seven samples (606 to 877 ohm-cm), but the chloride and sulfate contents are relatively low. The resistivity may be low due to presence of other ions, acids, or organic matter in the soil.

We performed mortar-coating testing and petrographic analysis. The results of testing showed mortar with normal unit weight, absorption and void content. The measured chloride content in the mortar indicated a corrosive environment at the M and A sites and a relatively low chlorides at the F and W sites. Close proximity to the roads and deicing salts are a potential source of chlorides at both the M and A sites.

The petrographic analysis indicated the following major findings:

- Overall the quality of the mortar coating is fair to good.
- There is evidence of deteriorated and altered paste in the outer surface of the mortar coating extending approximately 20% of the thickness in A pipeline, and up to 50% of the sample thickness in F pipeline. The alteration is most likely due to the combined effects of mild acid attack that resulted in partial dissolution of the near surface paste and carbonation (Figure 4).
- In the samples from pipeline A, we conducted Fourier transform infrared spectroscopy (FTIR) analysis on an extract of the surface paste and detected a single chain carbonyl layer indicative of localized patches of oily film that was present on the steel cylinder prior to application of the mortar coating.

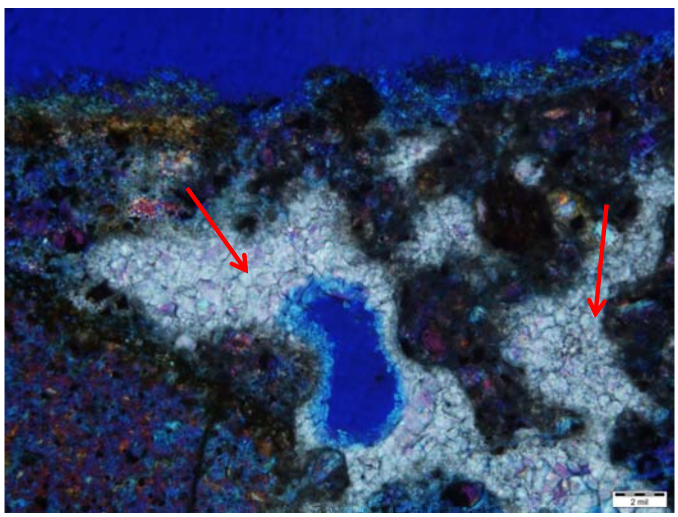

Figure 4 - In the near surface paste, we observe alteration of the cement paste and reprecipitation of secondary calcite (from the dissolution of the paste, appears white in photo) in air voids and pore spaces

A summary of our general observations at each inspection location, and discussion of potential effects on the average composite stiffness in pipeline segments measured by ePulse is provided below. Note that ePulse measures an average pipe stiffness over the pipeline length, and our external inspection and testing is limited to a small portion of the pipe within a segment length.

- A (24 in. diameter, Segment 3): We observed one broken prestressing wire, pitting on the surface of the prestressing wires, section loss of the steel cylinder, and a small hollow sounding area of the steel cylinder. We also observed that the prestressing wires on each side of the broken wire were loose, and may be broken under the mortar coating away from the opening. Mortar coating removed from the other openings exhibited poor bond to the steel cylinder, but we did not observe any hollow-sounding areas or cracks in the exposed portion of the pipe, including in the area of the corrosion and wire break. Our subsequent laboratory analysis determined that the poor bond between the mortar coating and the steel cylinder was likely caused by an oily substance on the steel cylinder.

Wire breaks and loss of tension in wires will significantly reduce the pipe stiffness. Poor bond between the mortar coating and the steel cylinder also lowers the pipe stiffness as the pipe wall is not composite in the debonded area.

- F (33 in. diameter, Segment 1): The exposed pipe did not appear to be distressed based on our external inspection. Our petrographic analysis identified alteration of the cement paste to a depth of 1/4 to 3/8 in. from the outer surface (40% to 50% of the cross-sectional thickness), most likely due to the effects of a mild acid attack. Degradation of the mortar coating may result in reduction in pipe stiffness and, therefore, reduction in composite stiffness from ePulse. Complete removal of mortar coating will reduce the hoop stiffness of the pipe by 22%; therefore, there are other factors that contributed to the reduction in composite stiffness such as pipe distress away from the external inspection location, variations in pipe properties or changes in fluid properties (e.g. entrained air).

- M (42 in. diameter, Segment 12): The exposed pipe did not appear to be distressed based on our external inspection. We identified corrosion of the bell ring at one end of the pipe. The steel bell ring was not well protected from the environment as the joint grout was loose and easily removed by hammer and chisel.

  Changes in ePulse composite hoop stiffness results may be due to potentially entrapped air at a high point in the pipeline, or other factors.

- W (16 in. diameter, Segment 16): The prestressing wires exhibited some pitting on the surface and the steel cylinder exhibited some section loss. We did not observe any hollow-sounding areas or cracks in the exposed portion of the pipe that would indicate distress.

  The changes in ePulse composite hoop stiffness may be due to pipe distress away from the external inspection location, variations in pipe properties, or changes in fluid properties (e.g., entrained air).

Conclusions

We conclude the following based on the available data and the work presented above:

- Based on the applications of ePulse to date, the technology appears to be a viable survey-level technology that, when coupled with structural analysis of the pipeline properties, can identify sections of pipeline with structural deterioration for further investigation.

- In the case study presented, the test segments with lowest composite stiffness identified by ePulse on each of the four pipelines were inspected externally. External inspection identified defects or operating conditions in three of the four segments that could explain the reduced wave speed. Pipeline A (Segment 3) had broken prestressing wire and delaminated mortar coating, Pipeline F (Segment 1) had alteration of the cement paste to a depth of 1/4 to 3/8 in. from the outer surface (40% to 50% of the cross-sectional thickness), and Pipeline M (Segment 12) potentially contained entrapped air at a high point in the pipeline.

- A desktop study consisting of failure risk analysis and structural evaluation can determine whether pipe rupture will be preceded by significant corrosion of steel cylinder and likely leakage, and whether section of pipe satisfy the requirements of AWWA C304 under the applied loads.

- The ePulse results are sensitive to changes in the wave speed, which may be affected by the unknown amount of entrained air in the sewage.

Alternative Construction Methods and Pipe Material Provide Solutions for Cleveland WWTP Project

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Abstract
The Easterly Wastewater Treatment Plant (WWTP), located on the northeast side of Cleveland between Lakeshore Boulevard and Lake Erie, provides wastewater treatment services for 334,000 residents and various businesses in northeastern Cleveland and the surrounding suburbs. In addition to treating wastewater from homes and businesses, the Easterly WWTP also receives and treats stormwater from combined sewers. Over 94 million gallons of wastewater per day (mgd) are treated at the Easterly WWTP. The Northeast Ohio Regional Sewer District (NEORSD) awarded the contract to Shook Walbridge Joint Venture. The Sustained Secondary Improvements Project includes expanding the Easterly Wastewater Treatment Plant's 330 MGD capacity to 400 MGD. The project included improvements to the plant's existing aeration tanks and final settling tanks, construction of six additional final settling tanks, improve hydraulics to support the capacity increase, and implement various miscellaneous improvements. More specifically, as part of the improvements, the project required auger-cast–piles to support new structure foundations, but also to support pipe cradles for the final effluent piping. These piles were part of the original design to support the 60", 72" and 84" diameter effluent pipe, which was designed with Prestressed Cylinder Concrete Pipe (PCCP). As a way to solve some constructability issues, Shook-Walbridge proposed a pipe material and pipe foundation substitution in lieu of the PCCP pipe and auger-cast–piles. Their substitution included the use for large diameter fiberglass pipe along with a continuous concrete ballast slab to address floatation concerns. As part of the substitution review process, various aspects of the alternative installation method had to be reviewed prior to acceptance of this alternative. This paper will review the different key components considered during the substitution review process.

PROJECT BACKGROUND

The Easterly Wastewater Treatment Plant (WWTP), located on the northeast side of Cleveland between Lakeshore Boulevard and Lake Erie, provides wastewater treatment services for 334,000 residents and various businesses in northeastern Cleveland and the surrounding suburbs. In addition to treating wastewater from homes and businesses, the Easterly WWTP also receives and treats stormwater from combined sewers. Over 94 million gallons of wastewater per day (mgd) are treated at the Easterly WWTP.

This facility was originally constructed in the 1930's and since has undergone various expansions and improvements over the years. Due to the continued growth of the Northeast Ohio Regional Sewer District (NEORSD) expansion of the Easterly Wastewater Treatment Plant Secondary System is required to increase the capacity of the facility by 70 million gallons per day. This will increase the capability of the Easterly Wastewater Treatment plant from 330 MGD capacity to 400 MGD.

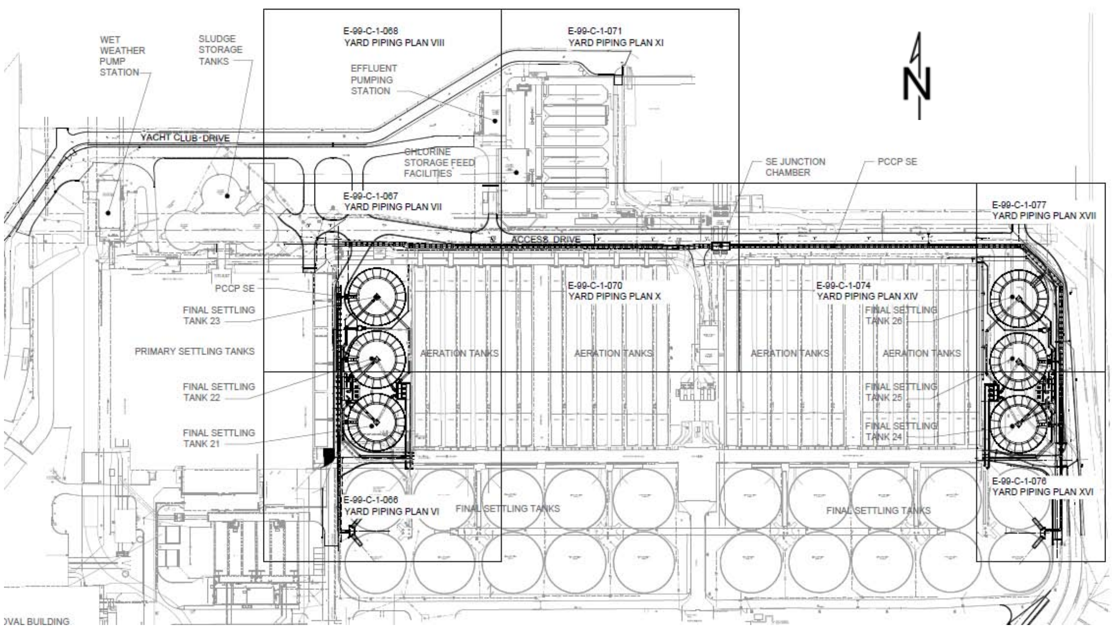

Figure 1 – Proposed Easterly Wastewater Treatment Facility Secondary System Improvements – Cleveland, Ohio

In order to accomplish this, the following list of major activities were required to achieve this capacity increase.

- Improvement to existing aeration tank
- Improvement to existing final settling tank Improvements
- Road Relocations
- Improvements to existing effluent pump station
- Construction of new final settling tanks
- Construction new conveyance conduit and channels
- Construction of new pumps stations

The Construction contract amount for this project is $ 74,350,000.

One of the most significant portions of the improvement to the Easterly Wastewater Treatment Facility was the work associated with the new secondary effluent tanks and conduits. New secondary effluent conduits were to be installed between the existing primary settling tanks and proposed final settling tanks (#21, #22, and #23). In addition, new conduits were to be installed on the northeast side of the proposed final settling tanks (#24, #25, and #26) and along the northwest side of the existing aeration tanks. The conduits would range from 48 to 84 inches in diameter.

In all, 6 new final settling tanks were to be constructed adjacent to the existing aeration tanks. Each tank, 105 feet in diameter, would extend 23 to 27 below grade, with the invert elevations of the conduits being 11 to 16 feet below grade.

SITE CONSIDERATION [1], [2]

Positioned on the south shoreline of Lake Erie, this site has gone through significant changes since its construction in the 1930's. Buried utilities and conduits are located throughout the site from decades of plant expansions and site improvement work. As part of the design process, a geotechnical investigation was performed to set a baseline of subsurface soil conditions, identify buried obstruction and establish ground water conditions which could be encountered during construction. This investigation involved subsurface exploration using traditional borings, laboratory tests, and field tests.

One of the most significant site conditions identified was the existence of fill material placed during the original construction and expansion of the facility. Fill material was used to build out the ground surface of the original shoreline in order to construct the plant and subsequent development further out into the lake. The majority of the existing structures within the project site were placed on timber pile foundations that were installed through the fill and into the underlying native soil.

One additional site condition to be considered was the groundwater surface elevations. Boring logs presented in the subsurface investigation report indicated that groundwater was encountered at the time of drilling in a majority of the structural borings at depths ranging from 8 to 48 feet below grade. The groundwater elevation generally corresponds to the water surface levels in Lake Erie. These levels may fluctuate due to precipitation and wind conditions.

Based on the subsurface site conditions it was recommended that the proposed new clarifier tanks and conduits greater than 60 inches in diameter be supported on a deep foundation system consisting of drilled shafts or 50' to 60' deep auger cast piles. Shallow foundation options were not utilized for these structures due to the potential for excessive hydrostatic uplift forces that could possibly develop beneath the structure foundations. Deep foundations systems consisting of driven piles were also not considered for support of the structures. Driven piles were not considered as the noise of the pile driving operations would be disruptive to the surrounding community.

DESIGN PHASE

The final design and layout of the proposed improvement to the Easterly Wastewater Treatment Facility included the utilization of Prestressed Cylinder Concrete Pipe (PCCP) for the new secondary effluent conveyance conduits. As previously noted, the pipe had to include a deep foundation system to support the pipe, and had to address buoyancy due to high groundwater levels. PCCP, a pipe material traditionally utilized by the NEORSD, appropriately addressed the pipe buoyancy concerns. The weight of the 48 inch through 84" PCCP materials allowed for the pipe to be installed using a traditional open trench method based on the following weights.

- 48 inch diameter PCCP – 900 lbs per foot
- 60 inch diameter PCCP – 1240 lbs per foot
- 72 inch diameter PCCP – 1780 lbs per foot
- 84 inch diameter PCCP – 2390 lbs per foot

The weight of the pipe was a benefit for buoyancy concerns, but required a deep foundation system to address the poor subsurface ground conditions and potential of long-term settlement of the proposed conduits.

As recommended in the Geotechnical Investigation, the PCCP was designed with a deep foundation support system to prevent the potential of long-term settlement. The supports were auger-cast-piles extending through the man placed fill from the original construction of the facility and into the native soils. These piles were designed with a concrete pile cap cradle for which the PCCP would rest. Figure 2 shown below details the pile and pile cap cradle formation.

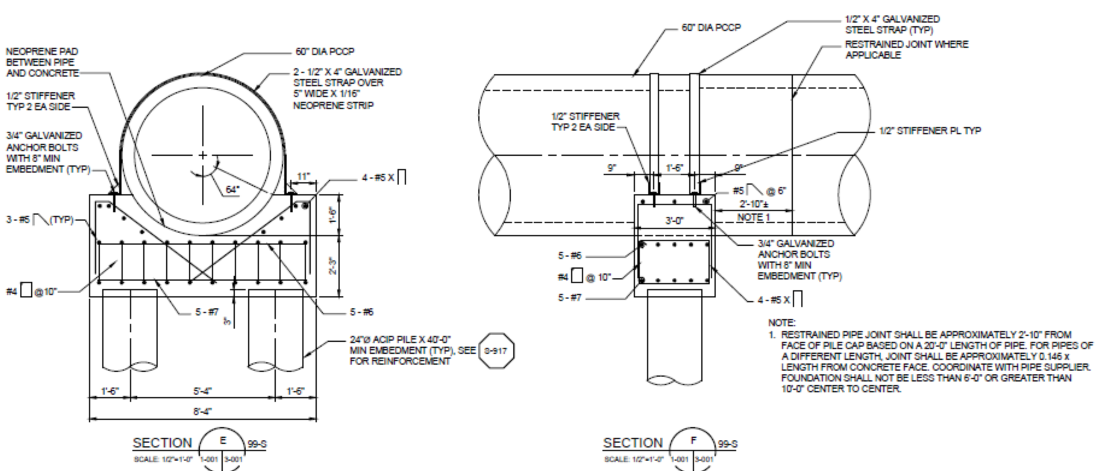

Figure 2 – PCCP Secondary Effluent Conduit Foundation Detail

The spacing of the foundations for the conduits took into consideration the weight of the pipe material, potential buoyancy, beam strength of the pipe in a buried installation intermittently supported on deep foundations. Based on all of these factors, the supports were spaced at a maximum of 10 foot intervals along the horizontal alignment of the proposed secondary effluent conduits. Figure 3 shown depicts a portion of the proposed deep foundation layout for the proposed PCCP.

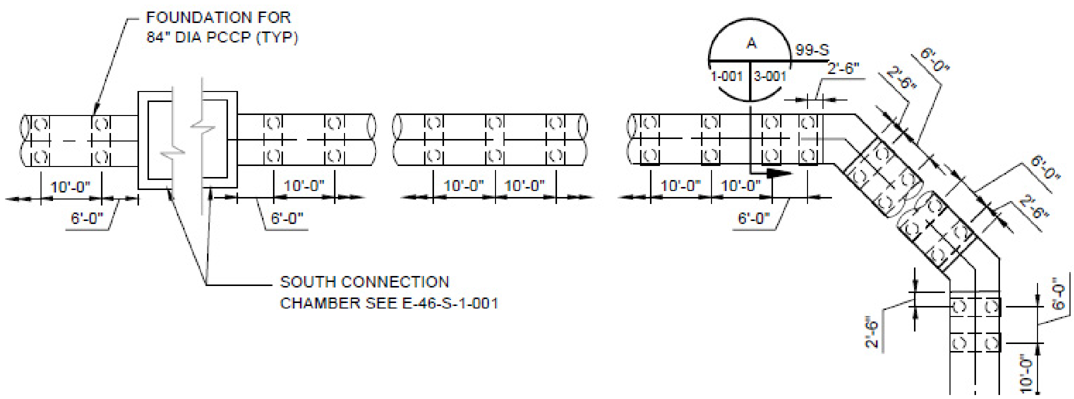

Figure 3 – PCCP Secondary Effluent Conduit Foundation Layout

The final design required auger-cast–piles to support new structure foundations as well as support the pipe cradles for the 60”, 72” and 84” diameter PCCP effluent piping.

CONTRACT PHASE

Base on the bid process, the Northeast Ohio Regional Sewer District (NEORSD) awarded the contract to the team of Shook Walbridge Joint Venture.To facilitate their proposed construction schedule, means, methods and a potential cost savings to the Owner, the Shook Walbridge team explored other pipe materials and construction methods to provide an alternate solution to the secondary effluent conduit piping and deep foundations. This alternate solution would still have to address the primary site considerations which were taken into account in the original design of the project. Thus the alternative would have to address the following:

- Differential settlement due to placement on man-place fill at the site
- Pipe buoyancy due to groundwater
- Pressure capabilities of pipe material
- Joint restraint of pipe material

Shook-Walbridge proposed an alternate pipe material and a pipe foundation substitution in lieu of the PCCP pipe and auger-cast–piles. Their proposed substitution included the use for large diameter fiberglass pipe along with a continuous concrete slab to address floatation concerns.

As part of the substitution review process, various aspects of the alternative installation method had to be reviewed prior to acceptance of this alternative. The proposed change could not affect the construction schedule or the specified warranty requirements of the contract. The added value for the owner over the PCCP is that the fiberglass pipe offers a longer anticipated design life and less maintenance given the resistance to corrosion, improved flow hydraulics, reduced cleaning and sediment build-up. In addition, this substitution offered cost savings to the project.

The contractor proposed the use of Flowtite filament wound fiberglass pipe manufactured by the Thompson Pipe Group. This product offers a minimum design life of 50-year in a corrosive sanitary sewer application.

The filament wound fiberglass pipe proposed weighed only 1/10th the weight of the PCCP. The weight of the 48 inch through 84" fiberglass pipe had the following pipe weights.

- 48 inch diameter PCCP – 127 lbs per foot
- 60 inch diameter PCCP – 179 lbs per foot
- 72 inch diameter PCCP – 247 lbs per foot
- 84 inch diameter PCCP – 424 lbs per foot

The weight reduction provided a benefit during the installation process as smaller equipment could be utilized to install the pipe material.

In order to utilize the lighter weight fiberglass pipe potential buoyancy risk would have to be eliminated. The proposed solution was to ballast the fiberglass pipe down to a reinforced concrete slab as shown in Figure 4. This design offered buoyancy resistance through the weight of the concrete but more importantly took advantage of the soil weigh outside of the pipe due to the extended width of the slab. This slab design would be continuous and be installed everywhere the 60 inch, 72 inch and 84 inch pipes were to be installed.

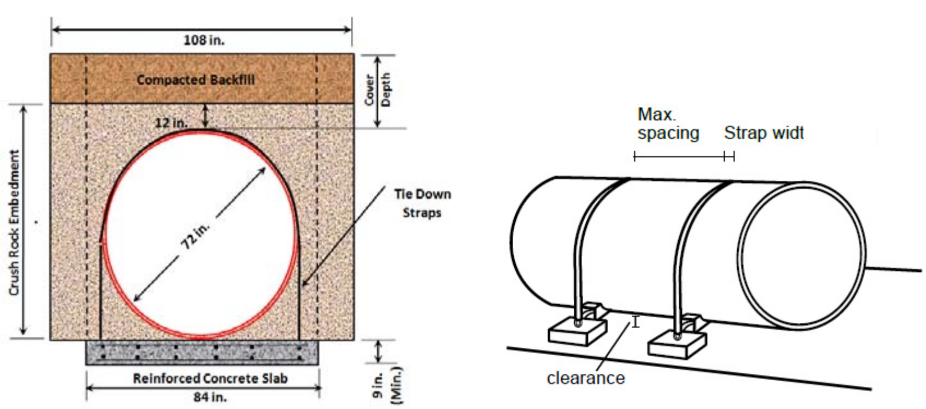

Figure 4 – Proposed Ballasting for Buoyancy Resistance

As the original design for the PCCP included harness restraints, the fiberglass pipe proposal also had to include the appropriate restraints where restraints were required. The proposal included three types on joint systems. The small diameter pipe (36 inch and 48 inch) would be restrained with a fiberglass key-lock system as shown in Figure 5. The larger diameter pipes (60 inch, 72 inch and 84 inch) would be restrained using a fiberglass or carbon fiber laminate on the inside of the pipe as shown in Figure 6. The third joint system was to address the sections of the effluent conduits connected to rigid structures. In order to offer restraint along with a flexibility to account for differential settlement between the rigid structures and the pipe, a harnessed style joint was offered with the Flowtite fiberglass pipe as shown in Figure 7.

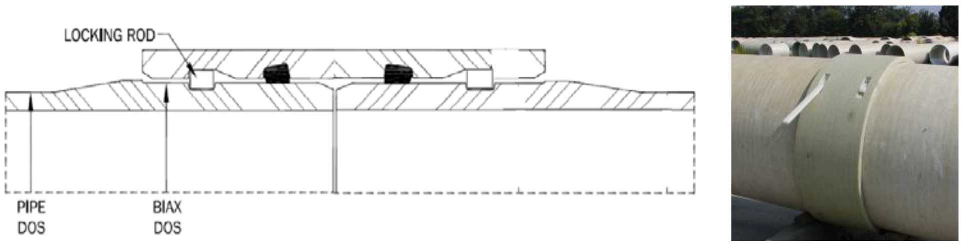

Figure 5 - Fiberglass "Key-lock" Restrained Joint System

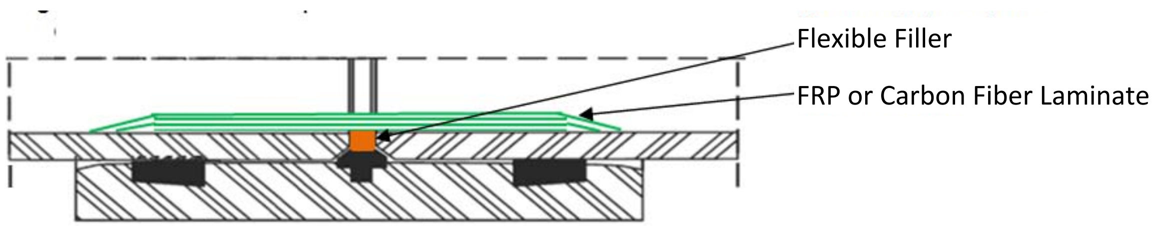

Figure 6 - Fiberglass Pipe Joint with Internal Laminate Restraint

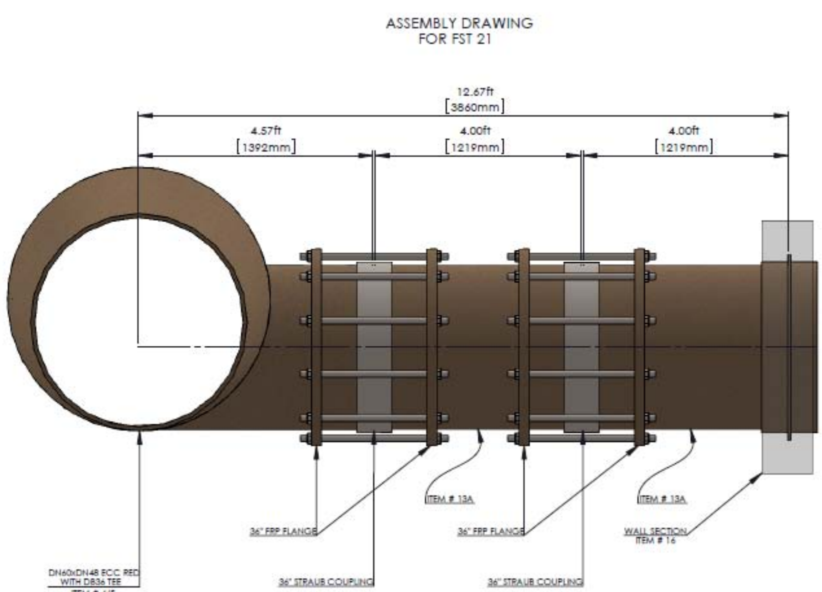

Figure 7 - Fiberglass Pipe Harnessed Restrained Joint

As this substitution was also changing the effluent conduit pipe material from a rigid pipe to a flexible pipe, other aspects of the design needing to be addressed included evaluating the fiberglass pipe. In accordance with the AWWA M45 Fiberglass Pipe Design Manual and specifications, this evaluation included the following:

- Pressure Class
- Working Pressure
- Working Pressure + Surge Pressure
- Pipe Long-term deflection
- Combined Loadings
- Buckling Pressures

In addition to the fiberglass pipe evaluation, the proposed substitution was reviewed by the Design Engineer and Owner to confirm that the alternative slab addressed the design issues previously discussed. With the design issues having been addressed, the substitution was allowed and the project was able to move forward into construction. The following is a summary of the benefits of this alternative pipe material and construction method:

- Significant cost saving to the Owner due to alternative pipe material and construction methods.
- Improved overall constructability of the project with respect to weight of pipe material and crane extension/capacity.
- Improved schedule of construction due to alternative concrete foundation design.
- Anticipated long-term maintenance and operation savings to the Owner with choice of corrosion resistant fiberglass pipe in a sanitary sewer application.

The construction of the secondary effluent conduit began at the start of 2015. The installation of the conduit will take approximately 5.3 months. Without the team work and effort of all parties involved, Northeast Ohio Regional Sanitation District, Brown & Caldwell, Shook Walbridge and Flowtite Pipe this project optimization would have never been possible. All parties involved helped to find a solution which was more economical, provided sound engineering solutions as well as provided long-term benefits to the Owner. This is the goal of every project but without a team effort his could not have been accomplished.

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Padre Island Water Supply Project Minimizes Environmental Impact Using HDD Technology

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Abstract

Padre Island is part of a barrier island complex that runs along the Texas coastline, from Galveston to Brownsville, Texas. It is bounded on the east by the Gulf of Mexico and by Laguna Madre on the west. A portion of the island is located within the limits of the City of Corpus Christi (City) and the water supply to the island is operated and maintained by the City. Prior to this project, the island was served by a 650 mm (24-inch) steel water transmission main that was constructed across the Laguna Madre in 1968. Survey and assessment of the existing pipeline indicated that it was nearing the end of its useful life. In order to increase the reliability of their water distribution system the City decided to pursue the design of a redundant water transmission main. To minimize impact to the natural resources in the area it was decided to locate the alignment on the south side of the John F. Kennedy Memorial (JFK) Causeway and to construct the pipe line using horizontal directional drilling (HDD) and conventional trenching methods. HDD was used to span the two major water crossings and conventional trenching was used to install the pipe through the dredged materials. During the design phase the City also decided to construct a 200 mm (8-inch) gas line and a 120 mm (4-inch) communications conduit along the same alignment as the water transmission main, allowing them to extend gas and communication service to the island. The final project consisted of 3,290 m (10,800 feet) of conventional trenching installation and 1,905 m (6,250 feet) of HDD installation for a 495 mm (18-inch) water transmission main, a 200 mm (8-inch) gas main and a 120 mm (4-inch) communications conduit. The first and most difficult HDD segment extended approximately 1,645 m (5,400 feet) from the mainland to the eastern side of the Humble Channel. A second 260 m (850 feet) HDD segment was constructed under the Gulf Intracoastal Waterway. This paper will discuss the design and construction of the Padre Island Water Supply project, focusing on the unique

project challenges presented by the close proximity to the coast and important natural resources, as well as the very long and challenging HDD sections utilized.

INTRODUCTION

Padre Island is part of a barrier island complex that runs along the Texas coastline, from Galveston to Brownsville, Texas. It is bounded on the east by the Gulf of Mexico and on the west by the Laguna Madre. A portion of the island is located within the limits of the City of Corpus Christi (City) and the water supply to the island is operated and maintained by the City (see Figure 1). The island is home to full time and seasonal residents with many dwelling units, hotels, condos and other rental properties. The JFK causeway is the link between the community of Flour Bluff on the mainland, and the community of Padre Island on the barrier island. The JFK causeway is comprised of dredged materials from the excavation of boating channels that crisscross the Laguna Madre. The JFK causeway separates the Laguna Madre on the south side from Corpus Christi Bay on the north side. The Gulf Intracoastal Waterway (GIWW) is a dredged channel cut through the east side of the lagoon along the west side of Padre Island. The Laguna Madre is a shallow salty lagoon that contains abundant sea grass beds. The Laguna Madre contains a large percentage of the sea grass that is found along the Texas Gulf Coast and the sea grass is protected by regulatory agencies like the U.S. Army Corps of Engineers (USACE), the U.S. Fish and Wildlife and the Texas Commission on Environmental Quality, among others.

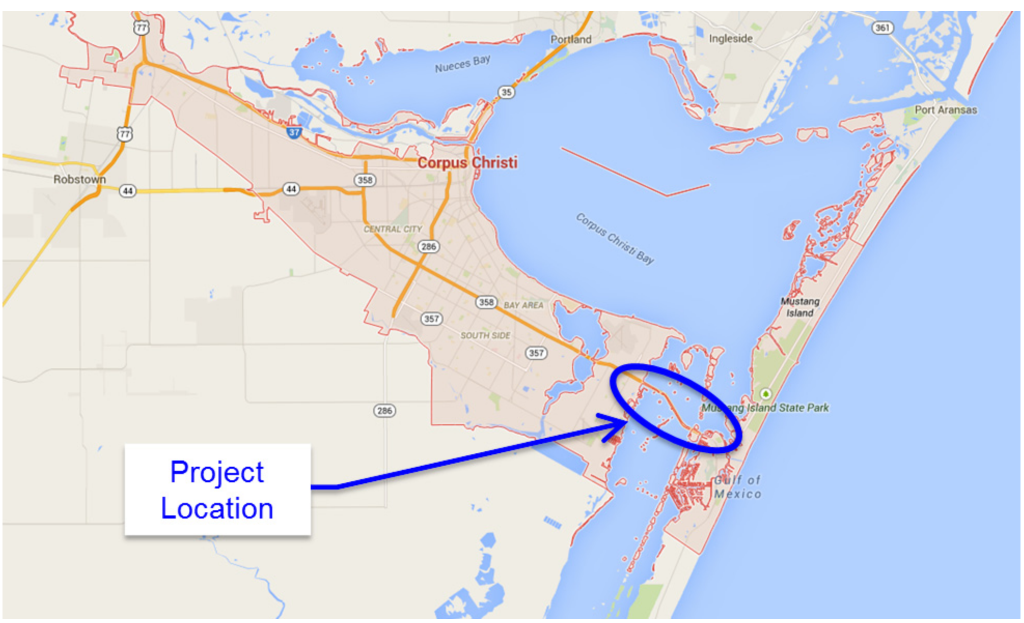

Figure 1. General location of the project shown along the JFK causeway crossing the Laguna Madre. Image courtesy of Google Maps (2015).

WATER SERVICE TO THE ISLAND

Historically, the City provided water to the island using a 650 mm (24-inch) steel water transmission main that was constructed across the Laguna Madre in 1968. Water is pumped from the City's water distribution system on the mainland to the Sand Dollar pumping station on Padre Island, using this 650 mm (24-inch) main. The Sand Dollar pumping station is then used to re-pump the water into the Padre Island water distribution system, where it serves the residential and commercial customers on the island, as well as providing bulk treated water to the City of Port Aransas.

In November of 2010, the City authorized a study of the water supply to Padre Island (Urban Engineering, 2011). The study looked at current and future water demands and modelled the existing Padre Island system to identify deficiencies. The study also assessed several long term and short term alternatives to supply water to Padre Island. At the time of the study, there was no secondary water transmission option to the island. If the existing 650 mm (24-inch) line were to be taken out of service or to fail, the City would have to rely on existing elevated and ground storage reservoirs to meet demand. According to the 2011 study, the existing reservoirs would only last 36 hours at average daily demand before the island ran out of water.

According to the hydraulic modeling done in the study, the capacity of the existing 650 mm (24-inch) water transmission main is adequate to meet the 2030 Max Daily demand. In 2010 the City also had an inspection conducted on the existing 650 mm (24-inch) water transmission main using an underwater CCTV camera and hydrophone sensor. The results of the inspection indicated that the existing transmission main was in good condition; however, the line is 46-years old with no cathodic protection, and is nearing the end of its theoretical design life. The transmission main lies buried in the sediment under the waters of Laguna Madre, so any future problems with the line will be difficult and costly to correct.

ALTERNATIVES ASSESSED

Two longer term and two shorter term alternatives were assessed in the 2011 study to create redundancy for the Padre Island portion of the system. The long term options included installing a second major transmission main across the Laguna Madre or routing a redundant supply of water through Mustang Island. The short term options included installing a smaller transmission main across the Laguna Madre, either along the JFK Causeway or through an existing, but decommissioned gas pipeline.

Due to the cost and availability of funding, the most economical short term option was to install a smaller redundant transmission main along the JFK Causeway. A 495 mm (18-inch) diameter PVC pipe was deemed appropriate for this installation at an estimated cost of $4.2 million. In order to increase the reliability of their water distribution system the City decided to proceed with the design of a water transmission main that would provide a level of redundancy and meet current and future water demands through 2030 if the 650 mm (24-inch) water transmission main had to be taken out of service or was to fail.

DESIGN AND PERMITTING

During the preliminary design phase of this project the City also decided to incorporate a 200 mm (8-inch) steel gas line and a 120 mm (4-inch) PVC communications conduit along the same alignment as the proposed 495 mm (18-inch) water transmission main, allowing them to extend and expand gas and communication services to the island.

The JFK causeway is located adjacent to important natural resources and one of the main objectives during the design phase was to minimize the effect of construction on these important resources. The area is mainly un-vegetated and the in-situ material is non-native and varies from unconsolidated pebbles to road construction materials. Where vegetation was present, it consisted of sea shore salt grass, sea oxeye daisy, and key grass, (HDR, 2012).

As with most linear utility projects that cross environmental resources and physical improvements, permitting is one of the key tasks that must be carried out. Due to the lead time required for review by the regulatory agencies, these tasks needed to be carried out as one of the first items in the design phase of the project. During the 2011 study, three agencies; USACE, Texas Department of Transportation (TxDOT), and General Land Office (GLO), were identified as requiring a permit.

In order to secure a permit from the USACE in a timely manner the project was designed so that it met the requirements of the USACE Nation Wide Permit 12 (NWP-12). The NWP-12 is the general permit for pipe line installation within the jurisdictional boundary of the USACE. A TxDOT permit was acquired for longitudinal installation of the utilities within the right-of-way of the JFK causeway. This was the first agency that the project was discussed with and the last one that received a permit application. In order to make sure that the environmental conditions of the other regulatory agencies were addressed by the design, TxDOT required that the City include an executed copy of the USACE preconstruction notification within its permit application package. Since the pipe alignment was located within the TxDOT right-of-way, the GLO only required that agreements to occupy various tracts with the lessee or with the GLO itself be secured.

UTILITY ALIGNMENT AND PIPELINE DESIGN

The water transmission main alignment was used as the centerline of the utility corridor. In general, the transmission main alignment for the project extends from the eastern edge of Flour Bluff at the intersection of the JFK causeway and Laguna Shores Drive to the western edge of Padre Island (see Figure 2). This portion of the JFK causeway follows along a narrow strip of dredged land that joins the Flour Bluff area of the City to Padre Island. The road is mainly built on dredged land but it also has two elevated sections and a bridge to traverse open water segments between the spoil islands. The north side of the causeway is narrower with less land available to construct the buried utilities as designed for this project. To minimize the impact to the natural resources it was decided to locate the alignment on the south side of the causeway and to construct the pipelines using a combination of horizontal directional drilling (HDD) and conventional trenching methods. HDD was to be used to span water crossings and conventional trenching was to be used where possible to install the pipe in the dredged materials (see Figure 2).

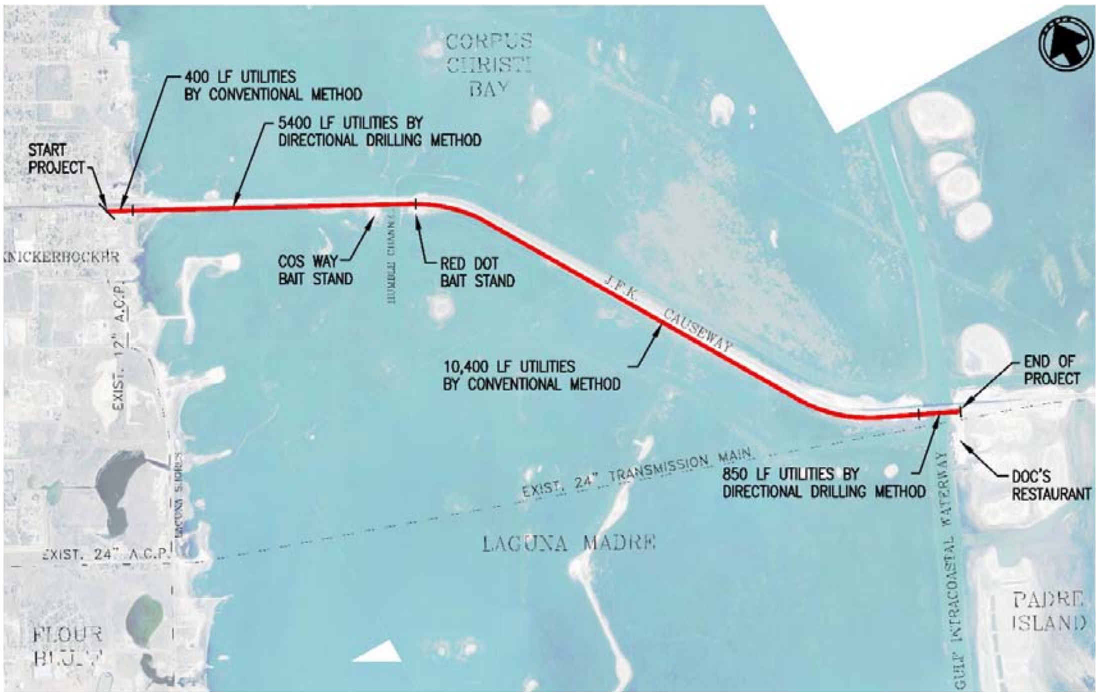

Figure 2. Overview of pipeline alignment and required construction types by segment.

Once the utility alignment was established, a draft profile was laid out to determine the location, geometry and configuration of the HDD segments. The longest water crossing segment was located between Laguna Shores and a point west of the Humble Channel. The strip of land west of the Humble Chanel is very narrow and becomes inundated with seawater during tidal events. The material in the area is fine, silty-sandy dredged material which does not lend itself to the operation of heavy equipment. The JFK causeway is elevated in this area by way of a retaining wall which also limits construction access. If the first HDD segment was to terminate in this area a more stringent general permit would need to be acquired from the USACE. The design team decided to relocate the HDD termination point further to the east, to the west side of the Humble Channel. The Humble Channel is a 30-ft deep waterway that provides small water craft access under the causeway to the Gulf of Mexico. There are tracts of land on each side of the Humble Channel that are owned and administered by the GLO and TxDOT. Several draft profile sections were reviewed and a conference was held with several HDD contractors to get their feedback on a preferred alignment. The drillers noted that if the drill alignment terminated at the western side of the channel, the drilling contractor would then need to set up and drill a separate alignment under the Humble Channel. The design team agreed that the most economical option would be to continue the initial drill all the way to the east side of the channel in one contiguous alignment. This meant that the total drill length of this first segment would be approximately 1,645 m (5,400 feet).

A detailed geotechnical study was undertaken during the design phase to provide geotechnical parameters for design and construction of the utility lines (Kleinfelder, 2012). A total of 16 soil borings, extending to depths of 4.5 to 26 m (15 to 85 feet) below ground were completed. According to the geotechnical report the project site is located on barrier island and beach deposits of the Beaumont Formation. These deposits are made up of fine-grained sand with some clay, silt and shell material. The borings indicated that the subsurface materials are comprised of alternating layers of granular soil, typically poorly graded sands and clayey sands, and fine grained cohesive soils, typically fat clays, sandy clays and lean clays.

HORIZONTAL DIRECTIONAL DRILLING DESIGNS

The general location and length of the HDD segments were established along with the pipeline alignment. The HDD alignment requirements also bracketed the pipe materials that were applicable to these particular HDD sections. Urban Engineering worked with potential drillers and pipe suppliers to define acceptable parameters for the HDD sections. For the longer, more challenging HDD alignment, a preliminary bore profile was laid out using a radius of approximately 305 m (1,000 feet). This bore had to pass under the navigable, Humble Channel. The Humble channel is approximately 7 m (23 feet) deep at the location of the crossing and is mainly used by small boat traffic. The preliminary profile utilized an entry angle of 6 degrees at Laguna shores and an exit angle of 9 degrees near Humble Channel. The depth of the preliminary profile was approximately 13 m (42 feet) to the top-of-pipe. Due to the geometry of the HDD profile, the 7 m (23 feet) deep Humble Channel is located very near to the end of the bore and the profile geometry was re-evaluated using a radius of 610 m (2,000 feet). This would minimize the amount of stress on the pipe as it is being pulled back through the bore hole. The larger radius would also increase the depth of the profile which would minimize the potential for inadvertent drilling slurry or fluid returns. Due to the length of the pull, drilling slurry or fluid returns escaping into the Laguna Madre was a major concern to the design team. The final HDD profile utilized a 10 degree entry and exit angle and a 610 m (2,000 feet) radius. This resulted in a total depth of 24.5 m (81 feet) to the top-of-pipe.

The second HDD alignment passed under the GIWW which is a commercial waterway subject to barge traffic. The GIWW at the proposed crossing was approximately 8 m (26 feet) deep. The preliminary GIWW bore profile also utilized a 305 m (1,000 feet) radius, with a 7 degree entry angle and a 10 degree exit angle. This bore profile was also adjusted and the bore radius was increased to 457 m (1,500 feet). In order to maintain the permitted entry and exit locations on either side of the GIWW, the entry and exit angles were steepened to 16 degrees.

Long HDD installations also require room to assemble and stage the pipe strings that are going to be installed. It is important to plan these areas in advance considering the required room to perform the work, stage the pipe, provide necessary access, and time the assembly per the other construction operations underway. During the design phase, two pipe staging areas for fusing and welding the pipe were developed. The staging areas for both crossings were located along the edge of the JFK causeway access road. The staging area for the Laguna Shores crossing was along the access road, east of the HDD exit area, and the staging area for the GIWW crossing was along the access road north of the entry point.

During the preliminary design phase the design team had considered allowing the contractor to bundle all three of the pipelines together into one bore alignment, or to allow them to install each utility in a separate parallel bore alignment. The final design allowed the contractor to choose which method they preferred, however, the gas line and water line could not be installed in the same bore alignment as the water line. The bid documents were set up so that each conduit could also be installed separately.

FINAL PIPE MATERIALS AND APPURTENANCES

Several pipe materials for the water transmission main were reviewed for use on this project including steel (AWWA C-200, 2012), HDPE (AWWA C-906, 2007), and PVC (AWWA C-905, 2010). Steel pipe was not considered to be competitive with plastic pipe at the required design size of 495 mm (18-inch), as it required a cathodic protection system and expensive coatings to withstand the harsh marine environment. At the time of the design, HDPE pipe material had not been proven as a viable material for long HDD installations as required for this project. Pipe buoyancy was also a consideration with the HDPE pipe and pipe collars would have been required to keep the pipe from floating during installation by the open trenching method. Since a large portion of pipe, 760 mm (30-inch) or less, being utilized by the City is PVC, the design team looked to use this same pipe material for this project. Fusible polyvinylchloride pipe (FPVCP) had a very successful track record for longer, deeper HDD installations, and had previously been installed by HDD in lengths greater than 1,645 m (5,400 feet). PVC pressure pipe was selected for this project as it is compatible with the City's current distribution assets, it is a material that the City can easily maintain and operate using their existing equipment, and it does not require cathodic protection.

According to the state, gas pipelines operating at greater than 414 kN/m^2 (60 psi) are considered to be operating at high pressure. The gas department decided to use steel pipe to satisfy state requirements for high pressure gas line installations within their right-of-way. The gas department also decided to install a passive cathodic protection system by their own volition at the completion of the project. The Municipal

Information System (MIS) department decided to utilize PVC SCH40 pipe for the conventional trench installation and AWWA C900 DR 14 PVC for the HDD segments of the (120 mm) 4-inch communications conduit.

Butterfly valves were installed for control and isolation on either side of the proposed HDD crossings of the water transmission main, in order to isolate the pipe segments from the rest of the transmission main for periodic testing purposes. Combination air release valves/air vacuum valves will also be installed at each side of the HDD crossings of the water transmission main to allow for the removal of entrapped air and to minimize water hammer from pipeline operations.

CONSTRUCTION

Bridges Specialties, Inc. (Bridges) was found to be the lowest responsible bidder with a total bid of $7.3 million. They were awarded the project and the notice to proceed was issued on April 4, 2014. Bridges elected to use the Mears Group to subcontract the HDD installation work. Bridges also subcontracted with Underground Solutions, Inc. to provide the fusion services for the FPVCP assembly.

The project was completed in a phased approach, with Mears and Underground Solutions, Inc. working on the two required HDD installation sections while Bridges supported them in those efforts. Congruent with this effort, Bridges also installed the open trench portions of the project when and where the coordination of site access and sequencing would allow them. Mears first worked on the GIWW crossing and then turned their attention to the longer Laguna Madre crossing.

Mears elected to install the three separate utilities in individual bore holes for each of the crossings, meaning that they drilled a separate bore hole for each utility at each crossing location, and pulled that utility through individually with no bundling. Completing the crossings in this manner assured them that after the first bore was completed, they could use the alignment of that first completed crossing to set their steering technology to accurately bore the other two at the appropriate offset. It also allowed them to deal with any issues during the installation of each segment on an individual basis instead of having to deal with a bundle of pipe materials and end uses.

GIWW Crossing

To minimize the impact on the environmental resources, the construction activity at the GIWW crossing was confined to a concrete covered area on the west side of the channel and to a 12 m by 24 m (40 x 80 feet) area on the east side of the channel. The bore entry and exit points were switched during construction to facilitate staging the pipe on the west side of the GIWW. The 140,000 pound drill rig was set up on the east side of the channel (see Figure 3). The drilling fluids were contained in a mud pit on the west side of the channel and large vacuum trucks were onsite during the drilling operations to remove the excess drilling slurry.

Figure 3. Drill rig set up for the first bore of the GIWW crossing.

The 120 mm (4-inch) communication conduit was joined by the butt-fusion process and after each joint had been completed, the internal raised bead at each joint was reamed and cut out using a debeading tool to assure a smooth inner diameter surface so that the cable, when pulled through, would not be damaged. The 200 mm (8-inch) steel gas pipe was placed on pipe supports prior to the start of the welding process. The pipe was welded and then x-ray tested prior to final installation. The pipe itself was 200 mm (8 inch) steel with a 14 mil coating of fusion bonded epoxy. An additional 40 mil coating was required for the HDD segments to protect the 14 mil coating during installation of the pipe. Once the welds had passed inspection the pipe joints were coated with compatible joint wrappers. The 495 mm (18-inch) pipe was placed on rollers as it was joined during the pipe fusion process. The rollers were spaced at required intervals to minimize the sag of the pipe and the rollers were used to move the pipe smoothly to reduce friction.

The pilot hole and reaming operations took approximately one to two days to complete for each bore. The pipe install for each bore was completed in approximately 1 ½ hours.

Laguna Madre Crossing

Prior to mobilizing the drilling equipment to the site, the contractor deployed silt fence around the exterior of the permitted pad sites. The drilling fluids were contained in mud pits at each side of the crossing and vacuum trucks were onsite during the drilling operations to remove the excess drilling slurry. A 1.3 million pound drill rig was set up on the Laguna Shores side of the crossing and the 140,000 pound drill rig set up on the Humble Channel side. Two drilling rigs were deployed so that Mears could drill the pilot hole from both sides if needed. The drilling operations began on the Laguna Shores side and progressed from this side in an easterly direction towards the Humble Channel for two weeks. As the drilling head advanced closer to the north edge of the Humble Channel Mears noticed that the pressure of the drilling mud was getting very high. In order to avoid an inadvertent fluid return into the Laguna Madre, Mears decided to stop their easterly advance and intersect the pilot holes by drilling with the smaller rig towards the west. Drilling from both directions reduced the length required to pump drilling fluid and subsequently the pressure of the drilling fluid in the formation. After the two drill bits met in the same alignment, the smaller rig backed out of the completed pilot borehole and the larger rig advanced the rest of the way to the east side of the Humble Channel.

The first installation was the 120 mm (4-inch) FPVCP communications conduit, which was initiated after the pilot bore was completed. The uncased installation of approximately 1,645 m (5,400 feet) of 120 mm (4-inch) FPVCP communications conduit represented the longest known installation of a thermoplastic pipe of this small of a diameter via HDD (see Figure 4).

Figure 4. Completed 4-inch FPVCP conduit installation by HDD.

A wireline was installed in the completed 120 m (4-inch) communications conduit to provide accurate guidance for the subsequent bore alignments. The second pilot bore was completed and a single ream pass was used to enlarge the hole and install the 200 mm (8-inch) steel gas pipeline (see Figure 5). Finally, the 495 mm (18-inch) bore was completed using two ream passes and a final 'swab' reaming pass to make sure the borehole was properly prepared. With the successful installation of the 495 mm (18-inch) waterline, the HDD portions of the project were completed (see Figure 6).

Figure 5. Insertion of the 200 mm (8-inch) Steel gas line into the Laguna Madre HDD section.

PROJECT RESULTS AND CONCLUSIONS

The project came in at budget with no unintended change orders and it was delivered in a timely manner without any schedule disruptions. The project was substantially complete on January 25, 2015 and the final walkthrough was completed on January 28, 2015. The City did not tie the 495 mm (18-inch) water transmission main into the exiting 650 mm (24-inch) transmission main on Padre Island as was intended in the design. The 495 mm (18-inch) water transmission main is temporarily connected to an existing 200 mm (8-inch) water main. The City has bid the second phase of the project that continues the line to Aquarius Street and they are currently in the planning stage of the third phase that will continue the new 495 mm (18-inch) transmission main alignment all the way to the Padre Island Pumping Station.

The 200 mm (8-inch) gas line is still waiting on final connections on either side of the installation. The gas department has bid the segment that will extend this line further east to Aquarius Street where it will tie-in to an existing 175 mm (6-inch) gas main that supplies the Padre Island gas infrastructure. The gas department is preparing to install a line to the west of the project site to complete the tie-in to their gas infrastructure in the Flour Bluff area.

The communication conduit is also not yet being utilized at the current date, but will eventually be used for fiber or communications cabling to Padre Island. Extension of the 120 mm (4-inch) conduit from the termination point of this project to Aquarius Street has been bid and the segment from Aquarius Street to the Padre Island Pumping Station is also in the planning stage.

One of the most important ‘lessons learned’ from a project of this nature is the importance of working with a contractor that has completed similar work and has demonstrated competence. If problems do occur, they will have the knowledge and skills to assess the situation and get the project back on track. In terms of the specialized HDD work on this project, this minimizes the possibility of inadvertent fluid returns and other material spills into the adjacent sensitive areas or waters of the state.

Figure 6. Insertion of the 495 mm (18-inch) FPVCP waterline into the Laguna Madre HDD section.

Another important item is to make sure that specifications contain the required language and/or bid items for the contractor to install silt fence or silt curtains, both along the conventional trenching segments and at the drill entry and exit locations. This requirement is especially critical at the drilling fluid collection pits. The drilling fluids rise and fall in the pit at different stages in the drilling operations. Make sure to require the contractor to provide adequate containment for the expected amount of fluid, such as berming the area around the pit when carrying out a project of this nature in an environmentally sensitive area.

The contract documents and project specifications also need to require the contractor to make arrangements to clean up and dispose of the drilling slurry, including the excess cuttings from the boring operations. They should also assure that these materials are disposed of at a facility that meets all of the state and federal regulations for such disposal.

Horizontal directional drilling saved time during the permitting of this project and it helped to minimize the impact of the project on the surrounding environmental resources. It also provided a clean means of installing a large footage of pipe in an area of very difficult construction. The importance of a competent design and construction team, which are well versed and fluent in the specialty installation methods and requirements of the project, is paramount.

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Water Mains Degradation Analysis Using Log-Linear Models

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Abstract

The development of reliable lifecycle intervention plans for water distribution systems depends on better understanding of water main degradation behavior. Traditionally, water main failures have been studied as Weibull/Exponential processes. This paper investigates the application of log-linear model for assessing metallic water main structural degradation. A comparison of the proposed model with the existing Weibull/Exponential based model is presented. Water mains inventory, operational and performance data from a Canadian municipality are used in the analyses. Conclusions concerning the adequacy of existing models and the applicability of proposed models are made. Municipalities and water utilities can use the method provided herein as a tool for desktop condition assessment and risk based failure analysis.

INTRODUCTION

Municipal water mains installed more than a century ago are still in service in many municipalities and water utilities across North America (Mirza and Haider, 2003). The condition and performance of water mains deteriorate over time because of a number of factors and complex processes that include, for example, mechanical, biological, and chemical degradation processes, environmental wear, water chemistry and operating conditions, accidental or intentional interference, defects during pipe manufacturing process, poor choice of pipe material and faulty design, poor installation and natural events. Ageing and deteriorating infrastructure systems along with the lack of maintenance have accelerated the degradation of these vital assets. A general assessment of the actual structural condition of these systems as well as analytical tools to assess their rate of deterioration are needed. The breakage history of the water main is considered as one of the performance measures by many municipalities and regulatory authorities. To determine the current condition of these assets, one must collect and assess data on the breakage histories of water mains. Unfortunately, many municipalities have only been rigorously recording breakage histories for a decade, while their pipes have been in the ground for much longer (Pelletier et al., 2003). To investigate the water pipelines failure, this paper uses the annual number of water main breaks as an indicator of the structural health of water distribution network.

To assess the performance of water mains, data on the breakage history of the water main network is considered as a useful indicator. Despite the fact that these assets are installed for a long time, many municipalities have a short breakage history of these assets. This study models the occurrence of breaks in metallic water mains by estimating: (1) the probability distribution of failure times; and (2) the number of pipelines failures depending on covariates, such as construction period, break time, and years in service.

The objective of modeling is to adequately reproduce the average tendency of the annual number of pipe breaks and to predict breakage rates in the future. This can help the municipalities and water utilities to ascertain present and future states and performance of their water distribution networks.

The data for this paper come from a Canadian municipality in Southern Ontario. The water distribution system consisted of metallic (ductile iron, cast iron, and steel), plastic (PVC, PE) and Asbestos Cement water mains. About 33% of the pipelines consist of cast iron. However, 95% of the breaks were recorded in cast iron water mains. Therefore, we considered only cast iron water mains in this paper.

WATER MAIN DETERIORATION MODELS

Deterioration models predict the future condition of infrastructure components and help decision makers to prioritize future maintenance, rehabilitation, and replacement activities. For the purpose of the deterioration modeling of water mains, information about breakage history is needed. For a thorough analysis of pipe breakage, information must be known on the physical and that have an impact on pipe failure (Pelletier et al., 2003).

To distinguish between the different orders of breaks, one must identify the time to failure between the installation and the first break, between the first and second break, and so on. This is called data stratification in survival analysis (Kalbfleisch and Prentice, 1980). Times to failure can be modeled by different distributions, depending on the breakage behavior associated with that break order. Eisenbeis (1994) used different distributions for different break orders and developed a calibration strategy, based on maximizing the likelihood function associated with the model. Such a calibration strategy extends the use of survival analysis to a much higher number of municipalities. The modeling and the calibration strategies are presented in Mailhot et al. (2000) and Pelletier et al. (2003).

Rate-of-failure (ROF) models and transition-state (TS) models are two types of water mains deterioration modeling developed in the literatures (Osman and Baindridge, 2011). ROF models interpolate the water mains breakage rate based on pipe age and environmental factors for a specific pipes segment without distinguishing the times between successive failures; whereas, TS models differentiate the time between successive failures of a specific pipe segment (Osman and Baindridge, 2011). The nonhomogeneous Poisson process model (Kleiner and Rajani 2010) and the Multivariate Exponential (MVE) models (Shamir and Howard 1979; Kliener and Rajani 2002) are some examples of the ROF models. Shamir and Howard (1979) introduced the first time-dependent Exponential model for forecasting water main break. They found that the rate of water main breaks increases exponentially with pipe age.

Gustafson and Calnsey (1999) developed and implemented (Gustafson et al., 2008) the transition state-life regression (TS-LR) to describe the time to failure between the first and second, the second and third, and so forth-breaks. These authors effectively showed that time to failure between breaks up to the 20th order can be described by Exponential distributions. Osman and Baindridge (2011) compared and analysed the ROF and TS models using a single data set for cast- and ductile-iron pipes in the City of Hamilton, Ontario, Canada. They concluded that the TS models rely upon a large and accurate historical water mains' breakage record, and enable municipalities to forecast future performance on the basis of multiple level-of-service standards such as breakage rates, number of breaks for any specific pipe, and probability of pipe failure in a defined time frame (Osman and Baindridge, 2011).

Wang et al. (2009) developed a logarithmic regression function to predict the annual breakage rates of water main. They found that there is a significant correlation between breakage rates, pipe diameter and length. Artificial neural networks (ANN) have been used to investigate deterioration of water pipelines by Achim et al. (2007); Al-Barqawi and Zayed (2008); Fahmy and Moselhi (2009); and Tabesh et al. (2009).

WATER MAIN DEGRADATION ANALYSIS

General description of case municipality. The water distribution network includes 501 km length of water mains, serving a population of 130,000. The following six characteristics are collected for all pipe segments: (1) pipe diameter; (2) type of material; (3) year of installation; (4) type of soil; (5) year of first break; and (6) year of the second to the fifth break. Table 1 provides information about the municipality and water distribution network.

Table 1. Characteristics of the case municipality.

Characteristics	
Population	130,000
Pipe network length (km)	501
Number of pipe segments in database	1,373
Number of pipe breaks in database	807
Year of installation of first pipes	1,850
Number of years of recorded pipe breaks	25

Weibull/Exponential model. The modeling strategy is used two distributions to model the different break orders. The Weibull distribution is associated with the first break order (time to failure from installation to first break), while the Exponential distribution is used to describe the behavior of subsequent breaks (time to failure from first to second break, second to third, and so forth). The time step for the time to failure is determined as one (year). The use of an Exponential distribution to describe the time to failure between the first and second, the second and third, etc. breaks is in agreement with the results of Gustafson and Clancy (1999). These authors effectively showed that time to failure between breaks up to the 20th order can be described by Exponential distributions. Moreover, they observed an almost constant parameter value after the fifth order**.**

The first step is fitting a probability distribution to sample data. Fitting a distribution to a dataset can be done by various methods such as Probability Paper Plot (PPP), Maximum Likelihood Estimation (MLE), and Moment of Method (MOM). The most versatile method used to analyze both complete and censored data is the MLE method.

At the time of analysis, a water main may have experienced the break starting the failure time under consideration, but the break ending the failure time has not occurred. This is known as a censored data, or a censor for short. Because the end of the failure time is not known, the data are said to be right censored. The value of the censor is the length of time that has transpired since the break starting the failure time.

Right-censored data are important as they contain valuable information about the survival. Omitting these data from statistical analysis could lead to an underestimation of life expectancy. The sample data are divided into two groups. The first group is completed lifetime and the second group is modelled as right-censored lifetime data.

To obtain the sample likelihood function given below, it is required to calculate the probability density function (PDF) of the complete lifetime data and reliability of the right-censored data.

$$L = \prod_{i=1}^{k} f(t_i) \prod_{j=k+1}^{N} (\overline{F} t_j) \qquad [1]$$

To compute PDF and reliability with Weibull and Exponential distributions, we can assume parameters of the distribution, and solve the problem numerically. The optimal solution is obtained by varying the value of unknown distribution parameters until the value of the likelihood function is maximized.

Weibull/Exponential model results and discussions. The Weibull distribution is associated with the first break order (time to failure from installation to first break), while the Exponential distribution is used to describe the behavior of subsequent breaks (time to failure from first to second break, second to third, and so forth). The Weibull distribution is defined by two parameters, $\boldsymbol{\alpha}$ and $\boldsymbol{\beta}$. The Exponential distribution is a special case of the Weibull distribution when α=1, with only one parameter, $\boldsymbol{\lambda}$. The $\boldsymbol{R^2}$ value from various types of probability paper plots is used to determine the "best" probability distribution for the data. The "best" fit distribution for the first break order of these data set is the Weibull distribution, and for the subsequent breaks is the Exponential distribution (R^2 is closer to 1, and the higher the R^2 value, the better the fit). Calibration parameters are presented in Table 2.

The Weibull and Exponential distributions PDF, cumulative distribution functions (CDF), and survival (reliability) functions are represented, respectively, in Equations 2 and 3 as follows:

$$f(t) = \frac{\alpha}{\beta}\,(\frac{t}{\beta})^{\alpha-1} e^{-(\frac{t}{\beta})^{\alpha}}$$
$$F(t) = 1 - e^{-(\frac{t}{\beta})^{\alpha}} \qquad [2]$$
$$\bar{F}(t) = e^{-(\frac{t}{\beta})^{\alpha}}$$

where, α is the shape parameter and β is the scale parameter of the Weibull distribution

$$f(t) = \lambda e^{-\lambda t}$$
$$F(t) = 1 - e^{-\lambda t} \qquad [3]$$
$$\bar{F}(t) = e^{-\lambda t}$$

where, λ is the scale parameter of the Exponential distribution

Table 2. Calibration Parameters of Weibull/Exponential Model.

Parameters	α	β	λ
	2.075	42.9	0.0381

First break. Probability density function with the Weibull distribution (time to failure from installation to first break) is shown in Figure 1a. The probability of first break from installation (time to failure) increases from time zero to its maximum value of 31 years, and then, decreases until reaches zero at the age of 100-year.

Survival function associated with the Weibull distribution is shown in Figure 1b. The value of the survival function gives the proportion of pipes that have not failed at time t. Therefore, the higher the curve, the longer it takes for the first break to occur, on average, in that municipality. The mean time to failure (MTTF) is equal to the area under survival curve and mathematically is shown in Equation 4. By definition (Stephens, 2012):

$$MTTF = E(t) = \int_0^{\infty} t.f(t)dt = \int_0^{\infty} \bar{F}\,(t)dt \qquad [4]$$

The mean time to failure associated with the first break is estimated to be 38.48 years. Alpha determines the shape of the distribution. Weibull distribution with α=2.075 exhibits an increasing hazard rate (Figure1c) at a constant rate (i.e. the probability of failure is increasing with time). Hazard rate is calculated as follow:

$$h(t) = \frac{f(t)}{1 - F(t)} \qquad [5]$$

Subsequent break. Probability density function with the Exponential distribution (time to failure from first to second break is shown in Figure 1a. The probability of second break decreases from time zero until reaches zero at the age of 100 years.

Survival function associated with the Exponential distribution is shown in Figure 1b. The mean time to failure for the Exponential distribution is equal to inverse of the parameter of λ. The mean time to failure associated with the second break is 26.25 years.

$$MTTF = \frac{1}{\lambda} \qquad [6]$$

The Exponential distribution hazard rate is constant and equal to the scale parameter, λ (Figure 1c). Therefore, a constant hazard rate of 0.0381 is obtained for the second break.

Figure 2 shows the MTTF between the first to the fifth breaks for the cast iron water main in the studied case municipality based on a 25-year breakage history data. The breakage history data obtained from the studied municipality were limited to the number of fifth break. The result shows that the MTTF is strongly related to the break number of a cast iron water main. It also indicates that after the fifth break, the value of MTTF is declining to zero.

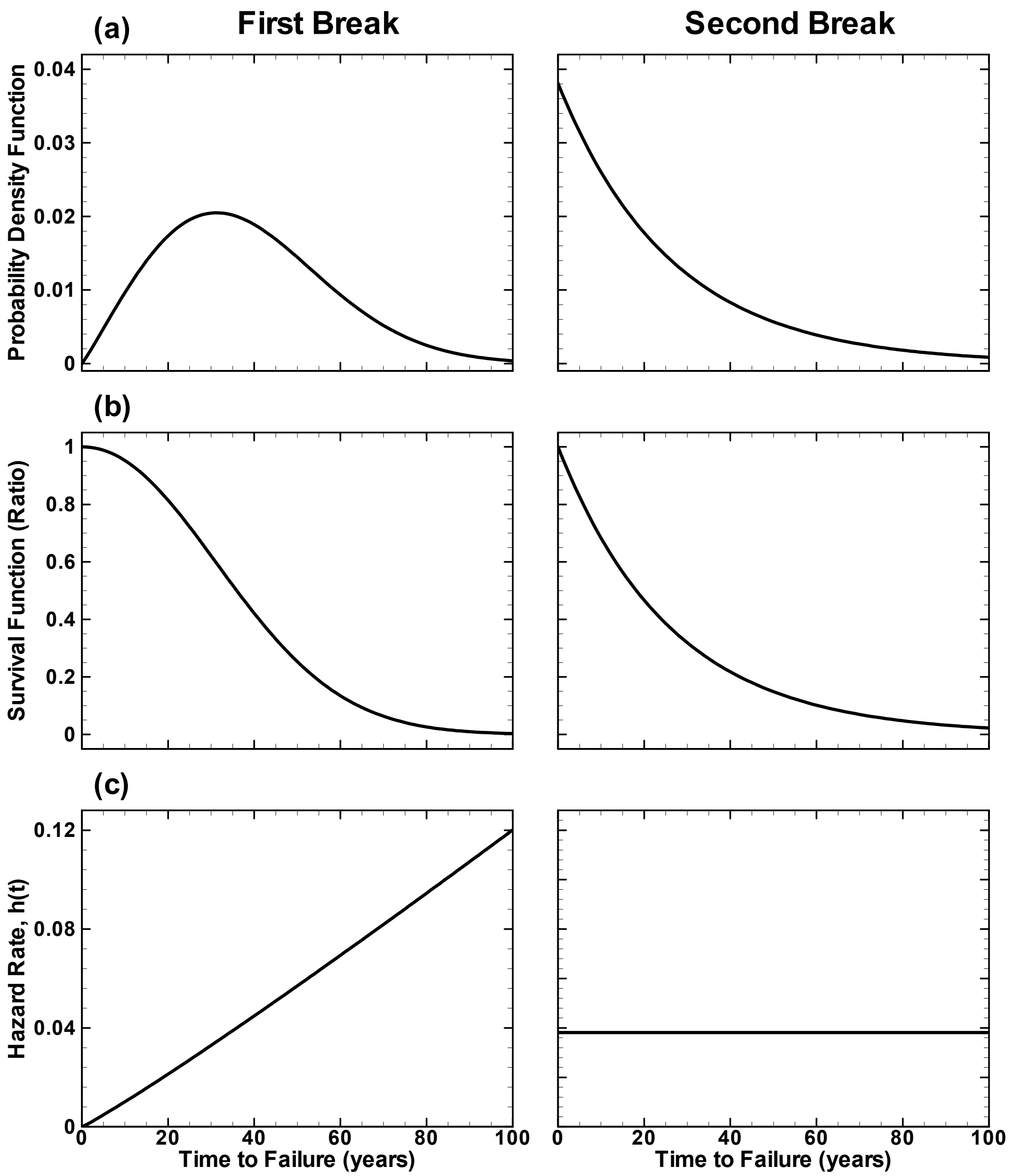

Figure 1. Weibull/Exponential model results for the first and second-break.

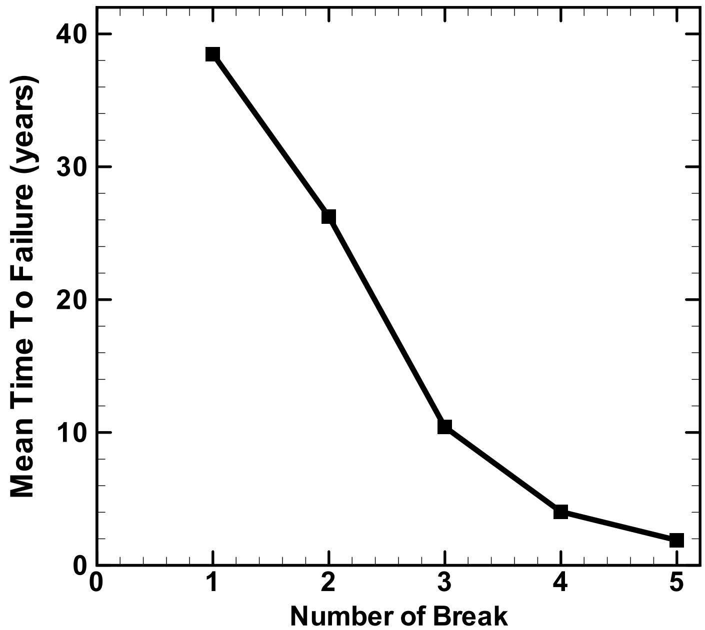

Figure 2: Mean time to failure vs. number of break

Log-Linear model. Log-linear model, also known as Poisson regression or Poisson log-linear model, is useful for modelling count or rate data (i.e., the number of events per unit time period). For given explanatory variables, $\boldsymbol{x}$, the log-linear model for expected rate is given as Agresti (2002):

$$log\left({}^{\mu_i}/_{t_i}\right) = \beta_0 + \beta_1 x_{i1} + \beta_2 x_{i2} + \cdots + \beta_p x_{ip} \quad [7]$$

Where ${}^{\mu_i}/_{t_i}$ is the expected value of the sample rate ${}^{y_i}/_{t_i}$. For water mains' break data, y_i is the number of water main breaks in a given time period, t_i. The explanatory variables, x, can include, for example, construction period, break observation period, pipe material, pipe diameter, soil type, location, and depth. Equation [7] can be written as:

$$\log\mu_i = \log t_i + \beta_0 + \beta_1 x_{i1} + \beta_2 x_{i2} + \cdots + \beta_p x_{ip} \quad [8]$$

Then, the expected number of breaks is given as:

$$\mu_i = t_i \exp\left(\beta_0 + \beta_1 x_{i1} + \beta_2 x_{i2} + \cdots + \beta_p x_{ip}\right) \quad [9]$$

For this study, Equation [9] is expressed as:

$$\mu_i = t_i \exp\left(\beta_0 + \beta_1(Construction\ Period) + \beta_2(Observation\ Period)\right) \qquad [10]$$

The maximum likelihood method can be used to estimate the parameters of the Poisson log-linear model as detailed in Simonoff (2003). The residual deviance, defined as the difference in the deviance of proposed model and the deviance of a saturated model, can be used for model checking and to perform goodness-of-fit test for the overall model as explained in Agresti (2002). The saturated model is a model that fits the data perfectly.

Table 4 presents the water mains break data in a contingency table where pipes installed during 1850 to 2010 are grouped along with the corresponding number of failures during the period 1986 to 2010. The table also shows the number of days the water mains were in service before the break. Table 5 shows the partial dataset where each case shows the observed number of breaks and covariates such as water mains' construction period, observation period, and days in service. For example, Case No. 29 shows that there were 68 water main breaks observed during the period 2001 to 2005 for water mains installed from 1951 to 1960, and water mains were in service for 17896 days. There are 50 such cases in total for the data presented in this paper.

Log-Linear model results and discussions. Table 5 shows the parameter estimates and deviance goodness-of-fit statistics. Note the days in service were converted to months before fitting the model. $\boldsymbol{\beta_0}$ is interpreted as the log expected count of breaks for pipes that were installed before 1901 and were observed for breaks during the period 1986-90. $\boldsymbol{\beta_{cyr(1951-1960)}}$ represents the ratio of expected number of breaks for pipes constructed in 1951-60 and observed in 1986-90 compared to pipes constructed before 1901 and observed in 1986-90. Figure 3 and Table 6 show that the predicted values are very close to the observed values confirming that the model fits the data well. This is further confirmed using the goodness-of-fit chi-squared test which is not statistically significant because $\boldsymbol{p-value = 0.1813}$ is greater than the usual significance level of $\boldsymbol{\alpha = 0.05}$.

For Poisson distribution, variance is equal to its mean. However, when Poisson regression is used on count data, as presented in this paper, variance can increase faster than the predicted mean. This phenomenon is known as overdispersion. To check for overdispersion, Figure 4 shows the standardized residuals versus the predicted values. The majority of points in Figure 4 are within two standard deviations (shown as horizontal solid lines at ± 2). Therefore, there is no evidence of overdispersion. The model accounts for 96.34% of deviance as shown in Table 5.

Table 3. Water Main Breaks Data.

Construction Period		Break Observation Period				
		1986-1990	1991-1995	1996-2000	2001-2005	2006-2010
<1901	No. of Breaks	8	5	1	3	1
	Days in Service	37254	33967	35793	37619	39446
1901-1910	No. of Breaks	4	7	4	7	0
	Days in Service	30315	32506	34332	36158	37985
1911-1920	No. of Breaks	5	3	5	3	2
	Days in Service	26663	28854	30680	32506	34333
1931-1940	No. of Breaks	2	3	11	4	2
	Days in Service	19358	21549	23375	25201	27028
1941-1950	No. of Breaks	6	10	7	14	4
	Days in Service	15705	17896	19722	21548	23375
1951-1960	No. of Breaks	48	53	61	68	17
	Days in Service	12053	14244	16070	17896	19723
1971-1980	No. of Breaks	13	34	25	40	13
	Days in Service	4748	6939	8765	10591	12418
1981-1990	No. of Breaks	0	11	16	11	2
	Days in Service	1095	3286	5112	6938	8765
1991-2000	No. of Breaks	0	0	11	16	4
	Days in Service	0	0	1460	3286	5113
2001-2010	No. of Breaks	0	0	0	1	0
	Days in Service	0	0	0	1094	2921

Table 4. Reorganized Water Main Breaks Data.

Case No.	Construction Period	Observation Period	Days in Service	No. of Breaks
1	<1901	1986-1990	37254	8
2	<1901	1991-1995	33967	5
…	…	…	…	…
6	1901-1910	1986-1990	30315	4
7	1901-1910	1991-1995	32506	7
…	…	…	…	…
10	1901-1910	2006-2010	37985	0
…	…	…	…	…
29	1951-1960	2001-2005	17896	68
30	1951-1960	2006-2010	19723	17
…	…	…	…	…
49	2001-2010	2001-2005	1094	1
50	2001-2010	2006-2010	2921	0

Table 5. Parameter Estimates.

	Estimate	Std. Error	z-vlue	$Pr(>\|z\|)$
$\boldsymbol{\beta_0}$	-5.71	0.25	-22.60	0.000
$\boldsymbol{\beta_{cyr(1901-1910)}}$	0.28	0.32	0.87	0.383
$\boldsymbol{\beta_{cyr(1911-1920)}}$	0.19	0.33	0.58	0.565
$\boldsymbol{\beta_{cyr(1931-1940)}}$	0.67	0.32	2.11	0.035
$\boldsymbol{\beta_{cyr(1941-1950)}}$	1.47	0.28	5.20	0.000
$\boldsymbol{\beta_{cyr(1951-1960)}}$	3.48	0.24	14.25	0.000
$\boldsymbol{\beta_{cyr(1971-1980)}}$	3.44	0.25	13.65	0.000
$\boldsymbol{\beta_{cyr(1981-1990)}}$	2.91	0.28	10.22	0.000
$\boldsymbol{\beta_{cyr(1991-2000)}}$	3.80	0.30	12.67	0.000
$\boldsymbol{\beta_{cyr(2001-2010)}}$	1.53	1.03	1.49	0.137
$\boldsymbol{\beta_{oyr(1991-1995)}}$	0.16	0.14	1.13	0.260
$\boldsymbol{\beta_{oyr(1996-2000)}}$	0.06	0.14	0.41	0.685
$\boldsymbol{\beta_{oyr(2001-2005)}}$	0.03	0.13	0.25	0.801
$\boldsymbol{\beta_{oyr(2006-2010)}}$	-1.44	0.19	-7.72	0.000

cyr: construction period; oyr: observation period

Null deviance: 1219.989 on 49 degrees of freedom

Residual deviance: 43.537 on 36 degrees of freedom, $\chi^2 \;\; p-value = 0.1813 > \;\alpha = 0.05$

Portion of deviance explained by the model = $1 - \frac{43.537}{1219.989} = 96.43\%$

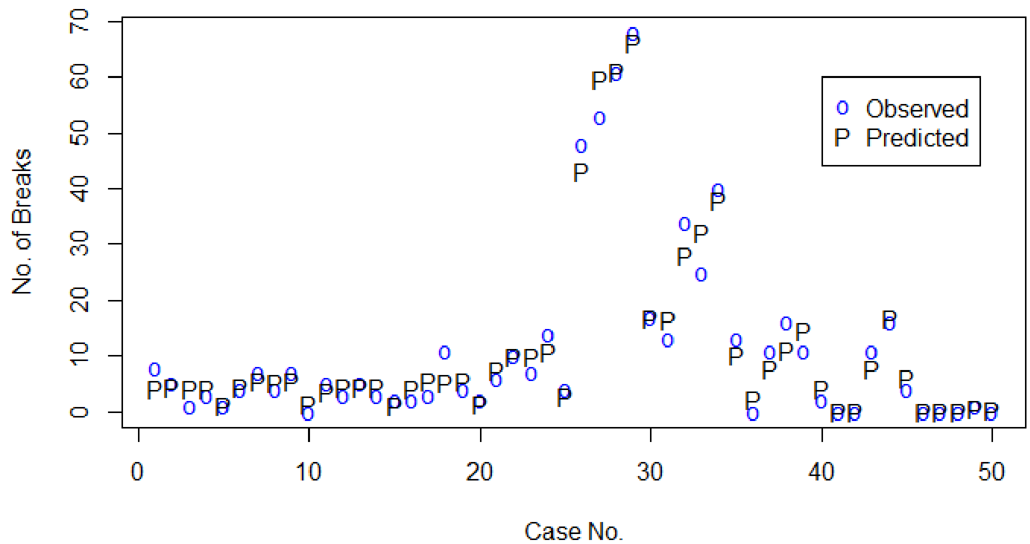

Figure 3. Observed (o) vs. predicted (p) breaks for the 50 cases (for case information, please refer to Table 4).

Table 6. Observed and Predicted Breaks.

Construction Period		Break Observation Period				
		1986-1990	1991-1995	1996-2000	2001-2005	2006-2010
<1901	Observed	8	5	1	3	1
	Predicted	4	4	4	4	1
1901-1910	Observed	4	7	4	7	0
	Predicted	4	6	5	5	1
1911-1920	Observed	5	3	5	3	2
	Predicted	4	5	4	4	1
1931-1940	Observed	2	3	11	4	2
	Predicted	4	5	5	6	1
1941-1950	Observed	6	10	7	14	4
	Predicted	8	10	10	11	3
1951-1960	Observed	48	53	61	68	17
	Predicted	43	60	61	66	17
1971-1980	Observed	13	34	25	40	13
	Predicted	17	28	32	38	10
1981-1990	Observed	0	11	16	11	2
	Predicted	2	8	11	15	4
1991-2000	Observed	0	0	11	16	4
	Predicted	0	0	8	17	6
2001-2010	Observed	0	0	0	1	0
	Predicted	0	0	0	1	0

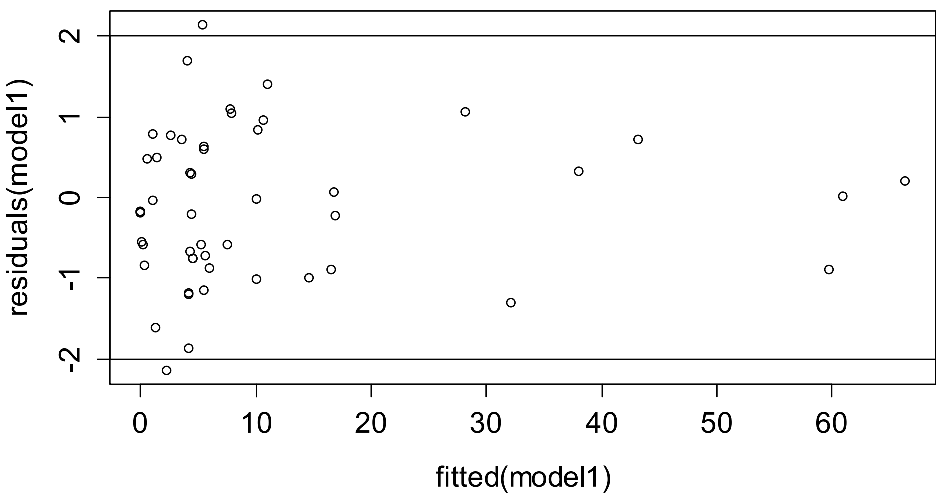

Figure 4. Residual plot.

CONCLUSION

The Weibull/Exponential process and Poisson log-linear model provide a useful framework in which to analyze the deterioration of cast iron water mains. Using the methods for analyzing survival or failure time data estimates of the probability distribution of failure times can be obtained. The analysis shows that in the case of years to first failure, the probability of failure starts out very low and then increases to a maximum before declining over many years. The MTTF is strongly related to the break number for a cast iron water main and decreases with increase of the break number. The results indicate that the MTTF of first to second break, second to third, and so forth versus number of break for cast iron water main is in agreement with the results of Gustafson and Clancy (1999). The authors effectively showed that time to failure between breaks up to the 20^{th} order can be described by Exponential distributions. Moreover, they observed an almost constant parameter value after the fifth order. Poisson log-linear model is used to predict the number of breaks per unit time for water mains installed over different time periods. The proposed models can be used as a desktop tool for water main failure analysis.

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Rehabilitation and Replacement of the East Layton Pipeline

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Abstract

The East Layton Pipeline is a critical aging potable waterline of the Weber Basin Water Conservancy District's (District or Owner) delivery system that serves as a major source of drinking water supply for the cities of Layton, Kaysville, Fruit Heights, and Farmington, which are located along the Wasatch Front just north of Salt Lake City. The original East Layton Pipeline was constructed in 1955 and includes segments of 30-inch-diameter bar wrapped pipe (AWWA C303) and nonstandard 27-inch-diameter centrifugally cast, reinforced-concrete pressure pipe with rubber gasket joints. Starting in 2010, the pipeline experienced an increased number of leaks requiring emergency repairs, which severely jeopardized the District's ability to provide a reliable water supply. The pipeline operates with limited system storage and could accommodate shutdown periods of no more than 24 hours during low demands. This paper documents the condition assessment of the existing pipeline, evaluation of alternatives for rehabilitation and replacement of the pipeline, and design and construction of the East Layton Pipeline using a Construction Manager/General Contractor (CMGC) arrangement between the owner, engineer and contractor. The rehabilitation and replacement evaluation resulted in replacing the existing pipeline with a 36-inch-diameter welded-steel pipeline on a separate parallel alignment through residential streets and slip lining the existing pipeline with a 24-inch HDPE pipe to provide system redundancy and ability to meet future demands. Key attributes of the project include construction of a major pipeline in narrow residential streets, crossing of a canyon with steep side slopes and potential landslide material, coordinating utility relocations, surge analysis of existing and future pump stations connected to the pipeline, easement and right-of-way acquisitions, and a significant public involvement effort prior to and during construction. This paper provides a detailed discussion of key issues and project challenges associated with the rehabilitation and replacement of aging infrastructure from the viewpoint of the Owner, Engineer, and the Contractor and also discusses the use and benefit of a CMGC alternative project delivery method.

INTRODUCTION

The Weber Basin Water Conservancy District's (Owner or District) drinking water system is relied upon by a number of cities and water agencies located in northern Utah. The District's East Layton Pipeline is the primary source of drinking water for multiple cities located along the populated Wasatch Front north of Salt Lake City, including the cities of Layton, Kaysville, and Fruit Heights. It is also a supply source for Farmington City. For some of these Cities this pipeline provides more than 90 percent of their drinking water and currently there is no alternate or redundant supply for many of the residences if this pipeline were taken out of service.

The northern 7000 foot long reach of the existing East Layton Pipeline has been experiencing an increasing number of leaks over the last 5 to 10 years which has increased operations and maintenance costs. Due to limited system storage, the pipeline cannot be taken out of service for more than 24 hours during low demand periods, making repairs difficult. Currently the pipeline is operating at design capacity during the summer months and is undersized for meeting projected future demands.

PROJECT HISTORY AND BACKGROUND

The original 7,000-foot reach of the East Layton Pipeline was constructed in 1955. At the time of construction, very few residences existed in the area, and the pipeline was constructed in the center of a 20-foot-wide easement that crosses Hobbs Ravine and then traverses open ground southeast toward Highway 89. The alignment was selected so that the pipeline would operate by gravity flow from the Davis North Water Treatment Plant (DNWTP) and to provide a relatively straight alignment across the open terrain. Figure 1 shows the existing pipeline alignment overlaid on a 1960 aerial image on the right and the same pipeline alignment overlaid on a 2012 aerial image.

Figure 1. 1960 and 2012 Aerial View of East Layton Pipeline

As shown in Figure 1, residential development subsequent to the original construction has greatly encroached upon the pipeline easement and some areas of the pipeline, including air vacuum and release vaults that are only accessible by foot through the backyards of homes.

CONDITION ASSESMENT OF THE EXISTING PIPELINE

In the mid 2000's the East Layton Pipeline started experiencing increased leaks and required more frequent repairs. In 2012 the District conducted a leakage test of the existing pipeline and hired a consultant to complete a condition assessment and alignment evaluation study to identify appropriate actions for repairing or replacing this critical facility.

Description of the Existing Pipeline

The majority of the existing pipeline is 27-inch nonstandard reinforced concrete pipe (RCP). The pipe is nonstandard in that it does not comply with American Water Works Association (AWWA) or American Society for Testing and Materials (ASTM) standards. The RCP was manufactured by centrifugally casting the pipe in a mold spun at high velocity and is often referred to as "Cenviro" pipe. The steel reinforcing in the pipe wall includes a thin wire mesh designed for internal pressures of 33 to 54 pounds per square inch (psi). The pipe was manufactured with standard flared concrete bell and spigot joints utilizing a single rubber-gasket. There is a section of pipe across Hobbs Ravine that includes approximately 1000 feet of 30-inch bar-wrapped concrete cylinder pipe (CCP) (in accordance with AWWA C303). The CCP was selected for this reach to handle the higher pressures (90 psi). The District has experienced only minimal leakage in the CCP and it appears to be in much better condition than the RCP.

Leaks

In 2012 the District performed a leak detection inspection of the existing line with a Sahara Leak Detection system which uses acoustics and video data to identify leaks and obstructions or abnormalities in the pipeline. A total of 15 leaks were identified as part of this survey. Two leaks were estimated to be between 75 and 128 gpm and the other 13 leaks were estimated at 2 to 75 gpm. All identified leaks were located at joints in the 27-inch RCP pipeline and approximately half were located near the Oak Lane Pump Station Turnout. The video system also captured locations of obstructions that included rocks and unknown debris in the pipeline. Some of the leaks identified were visible at the surface and were being monitored daily by District staff. Figure 2 shows two of the visible surface leaks located near the Oak Lane Pump Station turnout.

Surge Analysis

A surge analysis of the existing pipeline was performed to determine the impacts of starting and stopping the Oak Lane Pump Station that is connected to the East Layton Pipeline. The East Layton Pipeline operates at low pressures (less than 20 psi) and is located on the suction side of the Oak Lane Pump Station. The results of the surge analysis showed that the pipeline was experiencing significant down surges when the pump station is shut off and it is expected that many of the leaks at the pipe joints are a result of cyclical negative surge pressures wearing out the rubber gaskets. Figure 3 shows a hydraulic profile of the pipeline during a surge event. The red dots shown on Figure 3 show the location of the discovered leaks. Notice the location of known pipeline leaks relative to the down surges.

Figure 2. Pipeline leaks that surfaced near the Oak Lane Pump Station

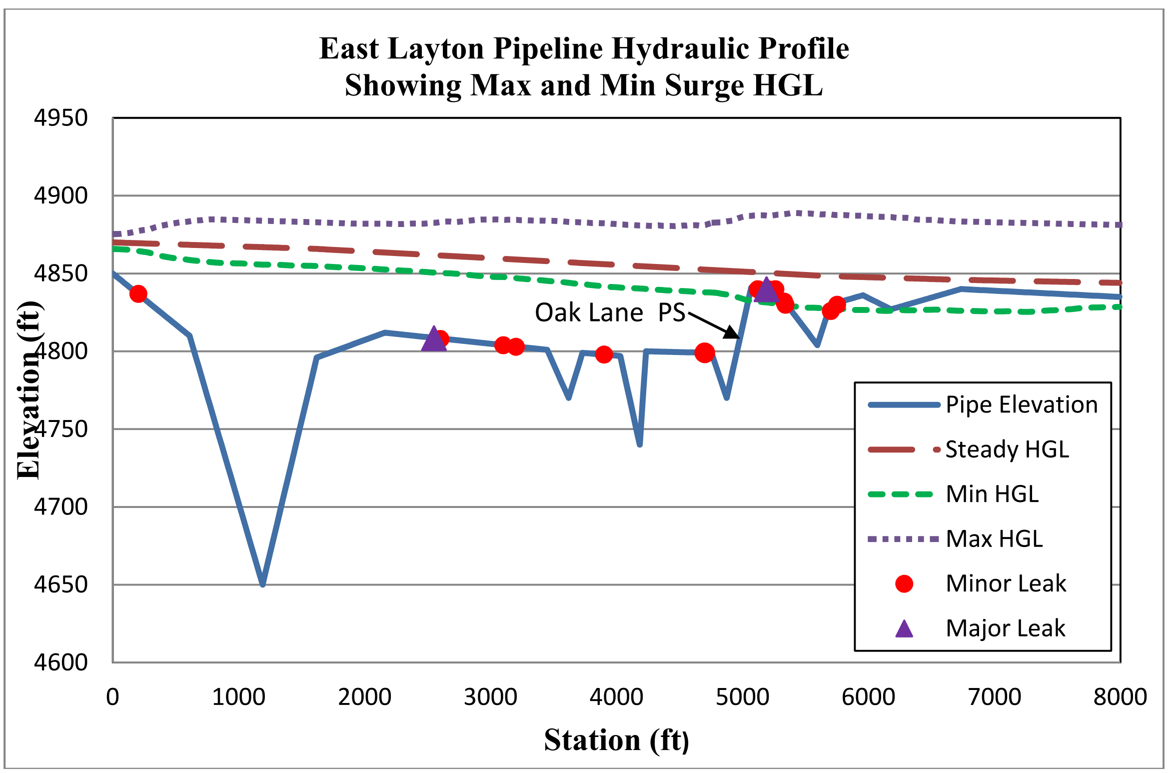

Figure 3. Hydraulic profile showing surge HGL and location of known pipe leaks

Repairs

The existing pipeline has been uncovered multiple times to repair leaks. In general the concrete pipe wall and cylinder is in good condition but failure of the pipe is occurring at the rubber gasket joints because of age, suspected pressure surges in the system, and tree root growth into the joints. The leaks have compromised pipeline function and added risk to the system and to the District. Historically temporary repairs have been made to the joints by tightly packing the exterior of the joints with grout or concrete encasing the joints. However this method has not always been successful and often serves only as a temporary fix. Figure 4 shows a joint of the 27-inch RCP being packed with grout during a leak that required repair in the spring of 2013.

Figure 4. Repair of concrete joint with packed grout

Figure 5- Slip-lining existing 27-inch RCP with 24-inch HDPE

After several unsuccessful attempts at repairing a leaky joint and having other leaks show up in the general vicinity shortly thereafter in the spring of 2013 the District was required to make an emergency repair to the pipeline which included slip lining approximately 1000 feet of the existing 27-inch pipeline with 24-inch outside diameter HDPE pipe. The District was able to install some temporary emergency cross connections with Layton City's distribution system which allowed the pipeline to be taken out of service for about a 5 day time period. This down time put a major stress on the rest of the water system and luckily the repair was made in the early spring when the pipeline was not operating at peak demands. The 1000 foot section eliminated 8 of the 15 leaks found during the leakage test conducted in May of 2012. Figure 5 shows slip lining the existing 27-inch pipe during an emergency repair.

Hydraulic Capacity and Increasing Demands

The existing pipeline was designed to provide a design flow of 8,400 gpm at the upstream end and 7,700 gpm at the downstream end. The pipeline operates under gravity flow conditions, with certain reaches operating at pressures less than 20 psi. These low operating pressures make it difficult for air valves to seat properly and are below the recommended AWWA minimum operating pressures of 20 psi for a drinking water pipeline. At these low operating pressures any down surges in the pipeline can cause the air valves to become unseated and then quickly slam closed when pressures are regained.

The 2012 peak hour summer demands in the pipeline were 8,050 gpm. Increasing flow rates in the pipeline will continue to reduce seating pressures at air valves and increase operation and maintenance (O&M) requirements. The District's forecasted 50-year demands require doubling the pipe capacity (8,050 to 15,600 gpm). Rehabilitation and replacement alternatives will need to consider the current and long term demand requirements.

REHABILITATION AND REPLACEMENT ALTERNATIVES

Following the condition assessment of the existing pipeline it was apparent that the pipeline could not be taken out of service for any extended period of time and rehabilitation/replacement alternatives investigated would require the installation of a separate pipeline on a parallel corridor that could at least meet existing and near future demands. The District established the following primary objectives.

- **Meet Long Term Demands:** The rehabilitation and replacement options considered must address the 50-year future (2060) peak hour demand of 15,600 gallons per minute (gpm). The 2012 peak hour demand was 8,050 gpm which is approximately the design capacity of the existing pipeline.
- **Increase System Reliability:** The East Layton Pipeline is a major drinking water source for Layton, Kaysville, and Fruit Heights and cannot be taken out of service for repairs during the peak summer months (June through September). During other times of the year the pipeline can be taken out of service for no more than a 24-hour period. A project objective is to increase the reliability of the District's water delivery system by formulating cost-effective ways to replace the existing pipeline and identify rehabilitation options for the existing pipeline that can improve system reliability and facilitate maintenance needs.
- **Reduce Operation and System Maintenance Costs:** The District desires to implement rehabilitation/replacement options and system improvements that reduce O&M costs and down time in delivery of water to the cities. The existing pipeline is near or at the end of its useful design life. The District has experienced frequent pipe leaks causing high O&M costs. Additionally, many of the air valves on the pipeline are located in the back yards of residential homes and are challenging to access. Pump station surges on the line are contributing to the leaks and are causing increased maintenance on air valves that seat and unseat when the pump stations shut off.

Because the existing pipeline can only be removed from service for 24 hour time periods, the District was interested in installing a parallel pipeline with the option to rehabilitate the existing pipeline in the future. Five separate pipeline alignments that provide a replacement for the existing pipeline were identified and are shown in Figure 6. Each alignment was evaluated against the Districts objectives along with present worth costs and other engineering factors. Table 3 provides the advantages, disadvantages, and present worth cost estimate of each alternative. A brief description of each alternative is provided below.

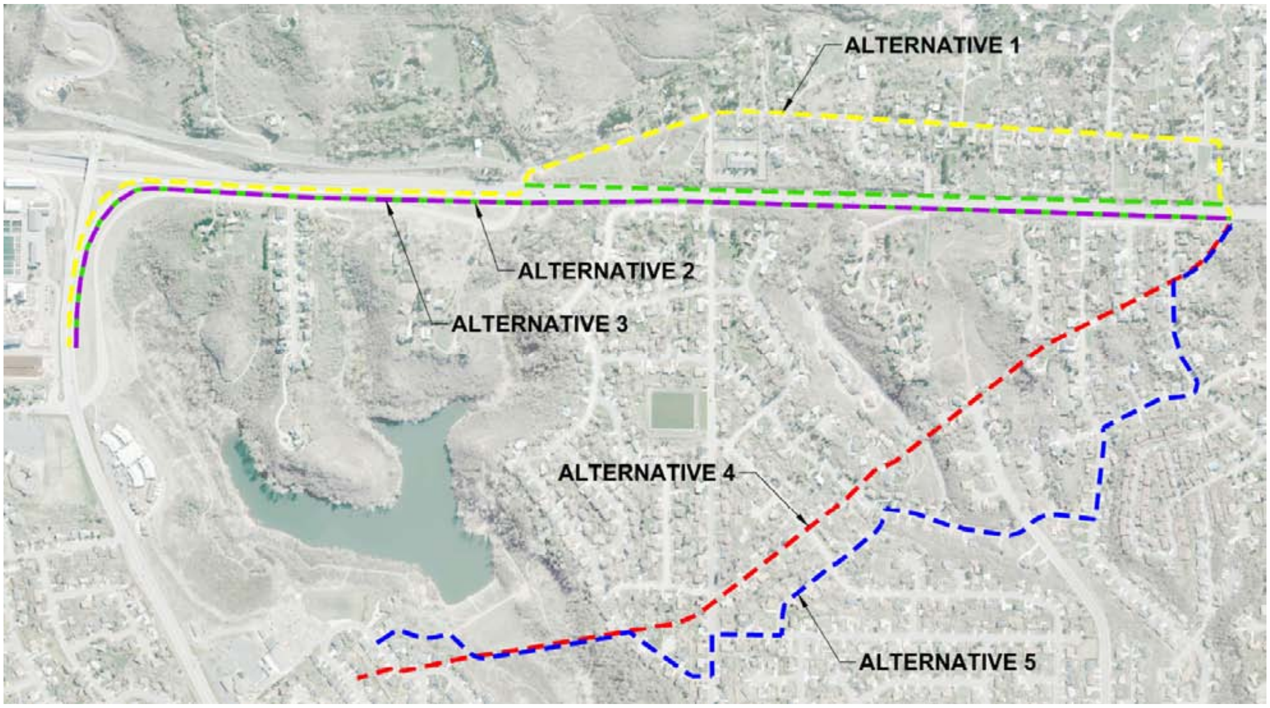

Figure 6. Alternative alignments for the Replacement of the East Layton Pipeline

Alternatives 1 and 2. Alternatives 1 and 2 were selected because they avoided residential neighborhoods and follow the alignment of a major state highway and frontage road. Portions of these alignments are above the hydraulic grade line of the DNWTP and require a pump station to be built which greatly impacts the capital and long term operation costs of these two alternatives.

Alternative 3. Alternative 3 shares the alignment of Alternative 2, but includes tunneling to avoid the need for a pump station. Tunneling represents a significant project risk and increased capital cost, but lower long term operating costs without the need for a pump station.

Alternative 4. Alternative 4 follows the same corridor of the existing pipeline. Because the existing easement is only 20 feet wide and significant development has occurred over and around the easement over the years, some segments of Alternative 4 would be tunneled to reduce the impact to local residents. This alternative includes significant challenges associated with the installation of a second pipeline parallel to the existing unrestrained and leaking pipeline, while keeping the existing pipeline in service. This presented significant challenges and risks.

Alternative 5. Alternative 5 was developed to provide an alignment that would eliminate construction in the backyards of residential neighborhoods and also allow gravity flow. This alternative requires construction of a large diameter pipeline in residential streets and disruption to the neighborhood.

TABLE 3
Advantages, Disadvantages, and Estimated Total Present Worth Costs of Alternatives 1 Through 5

Alternative/PWC	Advantages	Disadvantages
Alternative 1: Pumped Pipeline in Frontage Road **Present Worth Cost : $9,300,000**	- Limited disturbance in residential streets. - Completely separate route from existing pipeline, limiting risk of compromising existing pipeline, and stays away from residents.	- Pump station required with high O&M Costs - Future widening of Highway 89 could require relocation of the pipeline. - Construction congestion on major commuter route
Alternative 2: Pumped Pipeline in Highway 89 **Present Worth Cost : $7,600,000**	- Limited disturbance in residential streets. - Completely separate route from existing pipeline, limiting risk of compromising existing pipeline. - New pipeline would be out of the private backyards of residents	- Pump station required with high O&M Costs - Future widening of Highway 89 could require relocation of the pipeline. - Construction congestion on major commuter route
Alternative 3: Gravity Pipe/Tunnel in Highway 89 **Present Worth Cost : $8,300,000**	- Gravity flow pipeline - Limited disturbance in public streets. - Completely separate route from existing pipeline, limiting risk of compromising existing pipeline. - New pipeline would be out of the private backyards of residents	- Tunneling required with significant amount of unknowns related to cost and constructability - Future widening of Highway 89 could require relocation of the pipeline in the future. - Increased visibility and congestion on major commuter route
Alternative 4: Gravity Pipe/Tunnel in Existing Easement **Present Worth Cost : $6,900,000**	- Gravity flow pipeline - Shorter than Alternatives 1, 2, and 3 - Within the existing easement	- Disturbance in public streets and backyards of residents. - High risk of compromising existing pipe through construction and soil disturbance. - Tunneling with significant amount of unknowns and risk
Alternative 5: Gravity Pipe in Westerly Streets **Present Worth Cost : $5,900,000**	- Gravity flow pipeline - Lowest Cost Option - Pipeline located in existing easements or public street rights of way.	- Disturbance to residential streets. - Construction challenges of utility crossings in public streets. - Impact to residential neighborhood and will require significant public involvement effort.

Recommended Replacement and Rehabilitation Alternative

Based on a qualitative comparison of the alternatives, as well as a comparison of the present worth cost estimates, Alternative 5 is the best available alternative for installation of a parallel pipeline in combination with rehabilitating the existing pipeline. Key factors that led to the selection of Alternative 5 as the preferred alternative include: a reduction of risk during construction by avoiding construction within Highway 89 which would likely require some of the pipeline to be relocated when the highway is widened in the future, avoids tunneling, and provides the ability to operate the system through gravity flow (no pump station required). It was recommended that a 36-inch diameter pipeline be installed capable of delivering at least 10,000 gpm which will meet water demands for the next 10 to 15 years. This will allow the District to defer the cost of rehabilitation of the existing pipeline and provide additional time to take the existing pipeline out of service and further investigate rehabilitation options.

CONSTRUCTION MANAGER/GENERAL CONTRACTOR PROJECT DELIVERY

Because of the difficulty associated with construction of a large diameter pipeline located in residential streets, the increased leakage that was occurring on the existing pipeline, and the desire to accelerate the design and construction schedule, the District and their design engineer selected an integrated team approach to deliver the project by using a Construction Manager/General Contractor (CMGC) project delivery method. This allowed the Contractor to provide input during design, and suggestions related to design, cost, schedule, and constructability issues.

The project schedule was structured to allow multiple design activities to occur concurrently and keep the project moving forward at an efficient and accelerated pace. The contractual arrangement and benefits of the CMGC project delivery are discussed below.

Contractual Arrangement

Once the District was ready to move forward with the design of a new pipeline on a parallel alignment they hired a design engineer to prepare the final design and contract documents for the project. They also asked that the design engineer support the District with the procurement of a contractor to provide CMGC services during design which also included the option to move forward into construction if the pricing of the project was favorable. Figure 7 below shows the contractual arrangement between the Owner, Engineer, and Contractor.

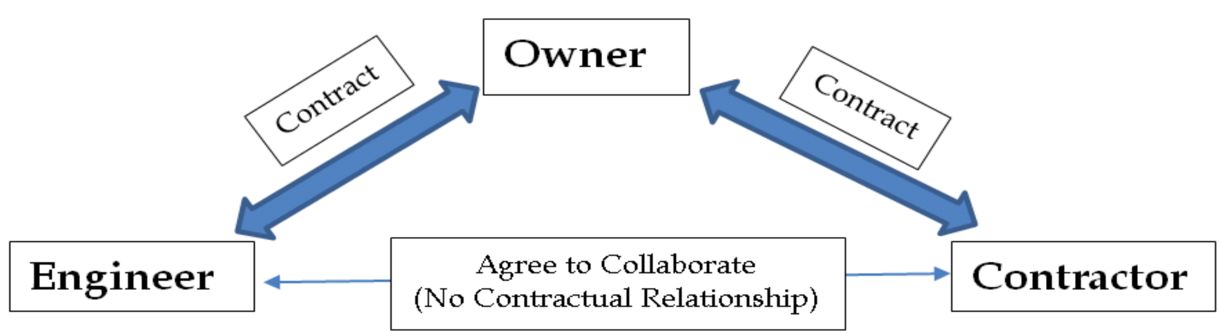

Figure 7. Contractual Arrangement of CMGC Project Delivery Method.

Using the CMGC delivery process the District was able to select a contractor during the design phase based on qualifications and best value. The CMGC was asked to provide design reviews at the 30, 60, and 90 percent design phases along with cost estimates at the 30 and 60 percent phase. At the 90 percent phase the CMGC was asked to provide an open book Guaranteed Maximum Price (GMP) so the District could determine if it was competitively priced. If the price was determined to be competitive then the District would move forward with a contract for construction. If the price was not competitive then the District reserved the right to terminate the CMGC contract and competitively bid the project similar to a traditional design-bid-build project.

Benefits of CMGC Project Delivery

Having a knowledgeable and experienced contractor on board during the design process proved to be very valuable. For a traditional design-bid-build project the design engineer cannot always predict the contractor's construction methods, plans for staging of equipment and material, proposed backfill material that will be used, plans for dealing with traffic control and surface restoration (paving), and many other items that cannot be established until the construction phase. With the CMGC delivery process many of these unknowns can be established during the design phase and gain endorsement from the Owner and jurisdictional agencies. A description of many of these items are summarized below.

Early and Consistent Public Involvement Team. The CMGC delivery method allowed the engagement of a public involvement team that started during the design phase and carried through construction. As part of the predesign phase the contractor was required to provide a public involvement firm that assisted the District and Design Engineer with several communication efforts including newsletters, brochures, public open house meetings, and one-on-one communication with affected residents. With the construction occurring in residential streets, the District knew that strong communications between project participants would be necessary for successful completion of the project. Being able to start the public involvement effort early prepared the community for the construction impact well in advance of any actual construction taking place. It also allowed the contractor to better understand project constraints such as coordination with school bus routes and residents requiring special needs.
During construction the CMGC provided a full time public involvement program that included a 24-hour hotline and a project website where residents could receive important announcements, view detour routes, and track progress of the construction. Figure 8 shows construction between two residential homes. Figure 9 shows construction in a residential street.

Figure 8. 36-inch Pipe Installation Near Residential Homes

Utility Investigations and Relocations. During the design phase the contractor assisted the design engineer with utility investigations by potholing and uncovering existing utilities. This allowed the contractor to have first-hand knowledge of the location of key utility crossings, better understand soil conditions that could be expected during trench excavation, and pavement conditions of the existing roadways. As part of the potholing and geotechnical investigations completed during the design phase, existing utilities on the pipeline alignment were marked in the field by utility locating crews. This provided the opportunity for the design engineer and contractor to meet in the field and discuss approaches to utility relocations and installing the pipeline at heavily congested utility crossings and intersections. Installing a large diameter pipeline in residential streets often requires utilities to be relocated and there are multiple pipeline alignments that can be selected depending on the utilities considered for relocation. Gaining agreement during the design phase between the contractor and engineer on approaches to dealing with existing utilities proved to be very valuable.

Backfill Materials. Based on the geotechnical investigations completed it was apparent that much of the pipe zone backfill material would need to be imported. This can represent a significant amount of the construction cost. During the design phase the contractor was able to identify local sources of material and provide submittals on the cost and characteristics of available materials. This allowed for the review and approval of backfill material, including unit cost, by all parties prior to construction.

Figure 9. Pipeline Construction in Residential Streets

Traffic Control. For pipeline projects designed in traffic congested roadways it is not uncommon for the design engineer to provide preliminary traffic control drawings, detour routes, and specifications that outline the contractors requirements and constraints for maintaining traffic during construction. This is primarily done to establish working limitations and traffic control requirements that will be enforced during construction. For a traditional design bid build project, the final details of the traffic control and detour plans are left for the contractor to negotiate with the jurisdictional agencies during construction. The CMGC delivery approach allowed the contractor to prepare traffic control plans and obtain the Cities approval prior to the award of the construction contract. This eliminated the pricing risk associated with traffic control and also provided early information to the public involvement team that could be used to help prepare residents for construction.

Negotiation and Review of Open Book Construction Costs. During the design phase the Contractor provided open book cost estimates at the 30 and 60 percent design phase and a GMP following the 90 percent submittal. The design engineer also provided independent cost estimating at each submittal phase. The District elected to not share the details of the engineer’s cost estimate with the Contractor but indicated to the contractor if they felt certain items were priced fairly or not and also indicated if they desired to reduce costs of major components of the project. The contractors open book costs allowed the District and their design engineer to see in detail how the contractor was pricing the work and provided opportunities to work with the contractor to identify potential cost savings as well as adding value to certain aspects of the project. Some of the cost saving ideas included backfill material selection, insurance and bonding requirements, and assigning final road restoration requirements to Layton City who had already established a low cost paving contract with another contractor. Added value items included the use of higher quality isolation valves, increasing the diameter and pressure class of the HDPE pipe installed at the Hobbs Ravine Crossing, and setting aside contingencies for unknown utility conflicts and relocations. Figure 10 shows the engineer and contractor's costs estimates during the three design phases.

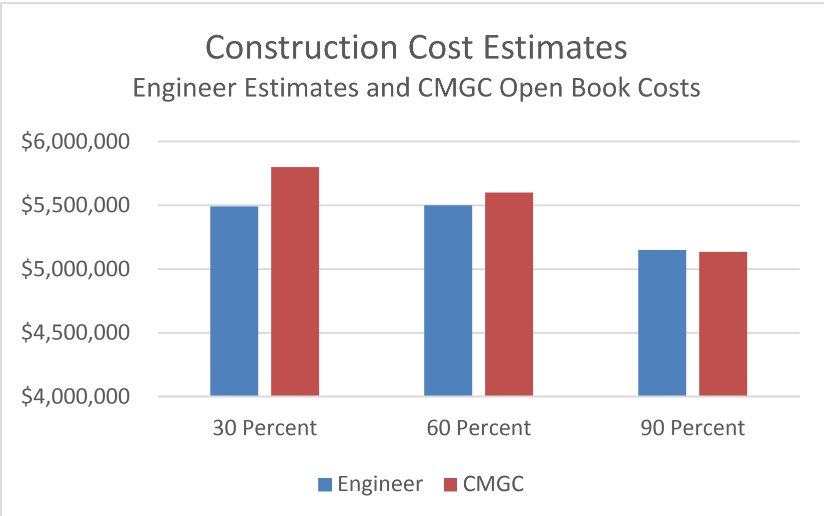

Figure 10. Engineer and CMGC Construction Cost Estimates

SUMMARY AND CONCLUSIONS

Many of the critical drinking water facilities installed along the Wasatch Front have reached or exceeded their intended design life and are starting to fail. Rehabilitating and replacing the East Layton Pipeline presented significant challenges due to the fact that the pipeline could not be taken out of service for extended periods of time. This represents a similar situation for most major pipelines that serve as the backbone of water infrastructure systems. Over the last 50 to 60 years residential development along pipeline corridors has greatly reduced the ability to access existing pipelines for maintenance, repairs, and replacement. Replacing the East Layton Pipeline on a new separate alignment allows the District to meet water demands for the next 10 to 15 years and the opportunity to take the old pipeline out of service for repairs and rehabilitation. Once the old pipeline is rehabilitated, the parallel pipeline system will provide the District with more reliability and redundancy to meet water demands for the next 50 years.

Using the CMGC project delivery approach the District was able to engage an experienced design engineer and contractor that were able to address significant project challenges associated with constructing a large diameter pipeline through residential streets. Together the project team was able to identify cost savings and added value ideas throughout the design and construction phase that allowed a quality project to be completed within budget and without major surprises or cost increases during construction.

Validating "Fully Structural": Development and Testing of a New Carbon Composite in situ Pressure Barrier for Trenchless Rehabilitation of Small-Diameter Pressure Pipelines

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Abstract

Trenchless technologies similar to Cured in Place Pipe (CIPP) continue to be embraced for pipeline repair and rehabilitation since the first installation in the early 1970's. The varying levels of repair from corrosion mitigation, leak protection, to semi-structural and fully structural repair systems require alternating levels of strength, stiffness and durability properties under loading conditions. ASTM F1216 is widely used in water and wastewater pipelines to determine the required thickness of composite liners for semi structural (class II and III) and fully structural (class IV) repair/rehabilitation systems as defined in AWWA M28 Appendix A. It is important to understand the initial assumptions and limitations of these design guidelines. The ASTM F1216 was developed for felt-epoxy CIPP systems that demonstrate quasi-isometric properties. Likely because of this, longitudinal loading is not considered in this design process. As CIPP products continuously develop to resist increasing external and internal loading conditions, stronger materials are used in specific orientations to meet those increasing demands. When unidirectional glass and carbon reinforced polymers (GFRP and CFRP's) are used to meet the demands of high internal and external loading conditions, additional design criteria are required to cover both hoop direction and longitudinal loading. These additional design criteria extend beyond ASTM F1216. Thus, theoretical calculations from existing and developing pipeline standards, and experimental validation is required to demonstrate the capabilities of new technology utilizing high strength and high stiffness materials like CFRP. This paper will address the additional design considerations appropriate for CFRP pull-in-place rehabilitations and the validation of a fully structural CFRP in situ pressure barrier for small diameter (six to fourteen inch) pressure pipe.

INTRODUCTION

This paper will cover practical considerations for CFRP applications for rehabilitation of pressure pipeline while looking at a few relevant design codes and guidelines and the assumptions made within these documents. When applications extend beyond the scope/initial assumptions of such criteria, the need for coupon, element and full scale testing becomes imperative to validate good engineering judgment. This paper makes the claim that assumptions written or implicit need to be well understood when designing, when installing, and when writing specifications/awarding a bid. Engineers, manufacturers, installers, inspectors and pipe owners cannot afford to waste time and money to install a product that does not effectively work. Providing an appropriate solution requires proper specification writing, design and validation through testing especially when new technologies are developed and brought to market. In addition, this paper will also discuss the process of validating the claim of a fully structural pipeline rehabilitation technology, in this case, a CFRP, in-situ, pressure barrier for small diameter pipe.

Prior to the development of CFRP trenchless technologies for large diameter pipe renewal and strengthening, CIPP products have provided efficient semi-structural and fully structural repairs. CIPP rehabilitation systems gained popularity for repairing and 'replacing' degrading pipe since the 1970's due to the ability to add to the capacity or take on the entire capacity of a pipeline without digging up the pipeline. This product originally consisted of a felt sock, impregnated with epoxy or other resins such as polyester or vinyl ester which cured after the system was pulled in or inverted into the existing pipeline which acts as a mold for the rehabilitation system. This application was originally for gravity lines with limited internal pressure. Because of its success, manufacturers expanded its capabilities, adding additional reinforcement in the hoop direction to take on increased internal pressures when necessary. This advancement has drastically increased the effectiveness of this technology, however design guidelines do not, nor can they be expected to, evolve as fast as developing technologies. Because of the rapidly degrading pipelines in America, a large array of technologies have been developed and implemented to rehabilitate water mains. Beyond the simple reinforcement of CIPP Liners with unidirectional CFRP or GFRP oriented in the hoop direction, many manufacturers/contractors have provided hand applied carbon and fiberglass composite systems when pipe diameters allowed manned entry and application, typically above 30" diameters. The small diameter CFRP rehabilitation technology represents the first pull-in place, CFRP in-situ pressure barrier and currently does not directly fall within the scope of recognized design guidelines and codes.

STANDARDS, CODES AND GUIDELINES

Engineering organizations like American Water Works Association (AWWA) have developed documents for design, installation methods and potential rehabilitation of water mains. *AWWA M28: The Rehabilitation of Water Mains* provides an overview of the process used for water main rehabilitation covering technologies with a proven track record within the water industry (AWWA M28 (Foreword) 2014). This currently is the only document that defines the structural classification of lining techniques, in Appendix A, which serves as the industry standard for defining a fully-structural or structurally independent liner. AWWA M28 has defined four classifications of liners: Class I for corrosion, Class II and III for partial internal pressure and spanning gaps and holes, and Class IV for the entire internal pressure with no strength of the host pipe. Because this paper will be focused on validating a fully-structural liner and the coinciding design considerations, the reader should peruse AWWA M28 for further explanation of the 1st three classifications. The fully structural definition is broken down into two main requirements from the technology:

"1. A long-term (50-year) internal burst strength, when tested independently from the host pipe, equal to or greater than the MAOP (Maximum Allowable Operating Pressure) of the Pipe to be rehabilitated

2. The ability to survive any dynamic loading or other short term effects associated with sudden failure of the host pipe due to internal pressure loads" (AWWA M28 2014)

The document then appropriately clarifies that although such "linings are sometimes considered to be equivalent to replacing the pipe, they may not be designed to meet the same requirements for external buckling or longitudinal/bending strength as the original pipe" (p112 AWWA M28 2014). This clarification can easily be overlooked and can lead to a fully structural repair that significantly relies on the strength and remaining performance life of the host pipe. AWWA M28 does not provide design guidelines for the Class IV lining but its reference to 'fully structural' implicitly references *American Society for Testing and Materials (ASTM) F1216: The Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing Of a Resin-Impregnated Tube*. This is widely used in the water industry as the design methodology for CIPP and other structural lining systems including fiber reinforced polymer technologies. However, the use of this design guideline far outreaches its scope and original intent.

ASTM F1216 was originally designed for resin impregnated flexible tubes for use in gravity and pressure applications for 4 to 108-in diameters (ASTM F1216 – 09). Resin impregnated felt is a quasi-isotropic material which has essentially equal strength in every direction. Likely because of this, the guideline only takes into account the hoop stresses on the pipe. Longitudinal or axial stresses like thrust, moment or thermal expansion/contraction and Possion's effects are not considered in this design standard. Although appropriate for the original CIPP technologies, the introduction of unidirectional FRP fibers in the hoop direction for pressure pipeline applications require additional considerations for design. This requirement for additional considerations becomes more obvious when unidirectional high strength

FRP technologies are used for fully structural designs without the use of resin impregnated felt at all. These FRP technologies can be seen in hand applied, CFRP rehabilitation of steel, PCCP, RCP, and many other degrading pipelines in America and across the world.

Although we must do so in an economic fashion, it is imperative to understand the life cycle behavior, environment and performance of a pipeline and its relationship with the rehabilitation technology. ASTM F1216 does not take into account the life cycle attributes but the design calculations provide a conservative approach to hoop direction loading for resin impregnated felt CIPP liners. Because unidirectional glass or carbon fibers are utilized to enhance the strength to thickness ratio of the rehabilitation technologies, it is imperative to understand the appropriate strength to consider. Although some carbon fiber systems are three times stronger than steel, the long term (50-year) design strain limits the effective strength to roughly 40% of their ultimate capacity based on physical testing (Xian 2008). This is based on Reiner-Weissenberg (R-W) criterion, a lifetime prediction analysis for sustained strains. ASTM F1216 does clarify that design properties must be time corrected for estimated duration of loading, however the strain compatibility with the host pipe, or strain behavior at all is not considered in this document. Typical rehabilitation technologies have a lower stiffness than the host pipe which allows the rehabilitation system to effectively share the load with the host pipe until the host loses structural integrity due to fatigue or corrosion. It appears important to recognize AWWA M28's second qualification for a fully structural liner, that the repair must survive any dynamic loading due to the failure of the host pipe. Although design guidelines provide these vital statements, such statements can proceed unenforced throughout the bidding process removing the effectiveness of the codes/guidelines. One reason ASTM F1216 has been implemented beyond its scope is due to the lack of a guideline or code that specifically covers unidirectional high strength, high stiffness fiber reinforced polymer systems for pipeline renewal in the water industry.

When F1216 was not implemented as the controlling design guideline on projects, CFRP systems were often designed based on the original code utilized during the initial design. As an example, for steel pipe rehabilitation, FRP was designed with AWWA M11: Steel Pipe, A Guide for Design and Installation with appropriate design considerations for the material properties of FRP. This approach can require owners and other parties to become subject matter experts and know what additional considerations should be required in the design and installation of these rehabilitation technologies which can be come thoroughly time consuming. However, when taking into account the importance of purchasing a product that effectively provides the desired renewal or strengthening of a degrading water line, parties involved should consider the concept of validating the claim of a fully structural composite rehabilitation technology. Such a validation requires good engineering judgment, understanding the base assumptions of various design guidelines and adjustment of the design requirements for the validated behavior of developed rehabilitation without current codes/guidelines. This is well known to many people in the industry such as AWWA, which has a committee developing a standard for CFRP Renewal and Strengthening of PCCP.

This AWWA draft standard utilizes an LFRD (Load Factor Resistance Design) approach for the CFRP renewal and strengthening of PCCP. CFRP has been well suited for PCCP due to the ability to spot repair PCCP sections that have identified wire breaks. This document provides a guideline for a 50 to 100 year design life for CFRP taking into account the entire life cycle and relationship to the host pipe for either a composite strengthening design or a stand-alone/structurally independent design.

It remains vital to understand the base assumptions of design considerations. PCCP is a semi-rigid pipeline that relies on prestressed steel wires to take on all internal pressures and significant external pressures in the hoop direction. The external loading of a semi-rigid pipeline is inherently different than a flexible pipe design (i.e. steel pipelines and CFRP rehabilitation liners). The effective soil loading on a pipe is based on the relationship of soil settlement relative to the pipe and the corresponding ability for the soil to help support the pipeline. Semi-rigid and flexible pipes behave fundamentally different under soil loading. If the host pipe fails due to external loading, it will likely fail at the hinge regions and then behave like a flexible pipeline where a CFRP liner now takes on all external forces of the host pipe. The AWWA M28 document only considers the internal hoop loading (maximum internal operating pressure) for a fully structural liner, however the external forces during dewatering of the pipeline may be the governing loading condition depending on the burial depth, water table depth, and soil modulus.

Although not currently published, the design considerations and load combinations included in the draft AWWA standard's design hold relevance as an example of better design considerations for anisotropic materials used in pipeline renewal and strengthening. Table 1 shows a direct comparison on some of the considerations among water industry, oil and gas industry and nuclear industry pipeline codes and guidelines.

It is also important to consider the determination of the material properties used for the design, short term and long term. The properties for all CFRP repair system's design properties should be found utilizing the appropriate statistical analysis. One effective method is a Weibull statistical analysis used and implemented on LFRD designs. The AWWA draft standard requires design properties to be the characteristic value based on the 80% lower confidence bound on the 5th percentile value of a specified value according to ASTM D7290: Standard Practice for Evaluating Material Property Characteristic Values for Polymeric Composites for Civil Engineering Structural Applications. This test method utilizes the Weibull distribution and allows for the material properties to be compliant with the base codes of LFRD design methodology (ASTM D7290 – 06).

Table 1: Guideline/Code Design Consideration Comparison

Design Assumption	*ASTM F1216*	*ASME PCC-2*	*ASME N-589*	*ASME B31.1*	*AWWA PCCP Draft*
Hoop Design					
-Working Pressure	X	X		X	X
-Transient Pressure		X	X	X	X
-Vacuum Pressure		X	X	X	X
-Traffic Loads	X	X	X	X	X
-Soil Loads	X	X	X	X	X
-Ovality	X		X	X	X
-Deflection Limits				X	X
-Combined Loading				X	X
Longitudinal Design					
-Poisson's Effect			X	X	X
-Temperature Effect		X	X	X	X
-Thrust Effect		X		X	X
Design	ASD	ASD	ASD	ASD	LFRD

VALIDATING A FULLY STRUCTURAL LINER

When developing a new product, appropriate testing becomes vital to effectively prove the theoretically or empirically developed equations for structural behavior. Because of the complexities of CFRP laminate design, and the typically simplified initial theories of pipeline design which assume materials are isotropic, it is crucial to reevaluate the design considerations with good engineering judgment and validate them through coupon, element and full scale testing. The remainder of this paper will highlight the process of validating the design of a fully-structural CFRP in-situ pressure barrier for small diameter (six to fourteen inch) pipeline.

This specific product is designed to be a fully structural repair system for small diameter pipelines. It utilizes a unique multi-axial hybrid fabric specifically designed for pipeline loading. The system utilizes a well-known, well tested epoxy that saturates a low profile carbon and glass hybrid fabric. The epoxy is certified as a building material after significant long term (10,000 hour) exposure testing under the ICC ESR AC125 (International Code Council Evaluation Services Report Acceptance Criteria 125). The Epoxy also has undergone long term performance testing utilizing the Arrhenius model for both tensile modulus and tensile strength. Gary Steckel's accelerated environmental durability testing of this manufacture's materials demonstrated a modulus retention exceeding 95% for any exposure temperatures tested up to 55˚C (Steckel 2015). The testing also demonstrated greater than 95% normalized tensile strength for exposure temperatures from 22˚C to 49˚C with low level degradation that increased with increasing exposure temperature. The projected strength degradation was less than or equal to 10% after 100 years of continuous exposure to moisture at temperatures up to 55˚C based on Arrhenius analysis of the testing data.

The major loading conditions and combinations were assessed and the fabric was designed to specifically orient the strength in the respective loading directions. The conceptual design has been proven based on testing of large scale, hand applied CFRP rehabilitation technologies. The small diameter application was initially proven viable with an installation run, then subsequent material coupon testing such as ASTM D3039, which tests for tensile properties, and ASTM D790, which tests for flexural properties, to meet the requirements of various codes and guidelines for the water industry pipelines.

One major flaw seen in other structural liners was the presence of significant wrinkles. Surface wrinkles of the coating layer of structural liners are not a structural concern. However, if the reinforcing structural material layer has a fin which produces a discontinuity in the path of loading, significant strength reduction results and a premature catastrophic failure is likely. The ability to install a fully structural liner without wrinkles ensures the installed product fulfills the design properties. This particular systems expands and conforms to the host pipe eliminating the presence of wrinkles for straight pipeline. The installation method used builds confidence in the initial coupon testing. Because of the industry's use of ASTM F1216, the manufacturer performed significant flexural testing on coupons as well. To meet industry needs, the system was developed to cure in as little as two hours. Curing profile testing was developed and validated for the product and the coupons were manufactured with the same cure profile to further validate the actual performance of the installed product.

Because the coupon tests performed, ASTM D3039 and ASTM D7290, provide a method to validate fundamental material properties like tensile modulus, E, and do not necessarily take into account he triaxial state of stress that will occur in the rehabilitation technology, element and full scale testing was implemented to test both internal and external loading, along with service connection testing. To validate the full scale stand-alone performance of this pipeline rehabilitation technology, the manufacturer produced nominal 8" diameter samples for short-term burst testing in accordance with ASTM 1599-99: Standard Test Method for Resistance to Short-Time Hydraulic Pressure of Plastic Pipe, Tubing, and Fittings. The test is an axially restrained short term burst that mimics worst-case pipeline loading in the hoop direction without any strength of the host pipe. Figure 1 below, shows an image of a test specimen.

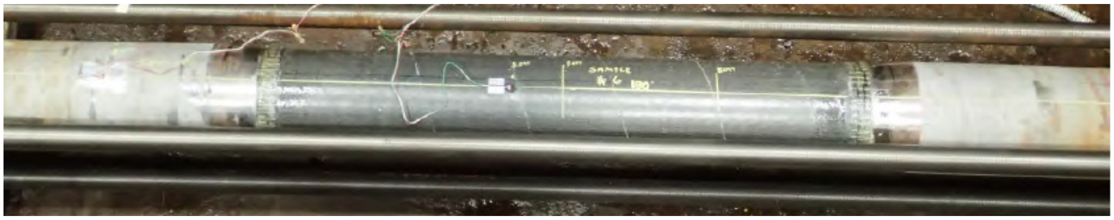

Figure 1: 8" diameter CFRP rehabilitation technology test specimen in axially restrained loading apparatus.

Because of the successful short term average burst strength of 1160+/- 90psi for 7 pipe samples installed utilizing the same installation technique and proven cure regime, the manufacturer performed two 300 ft trial runs (Sheets 2014). This full scale installation process was implemented to validate the ability to install the product at the scale required for market use and to test the actual pipe properties during installation. One 300 ft run was installed into Sonotube® (a cardboard tube acting as a mold that could be removed after installation), allowing for testing on the CFRP rehabilitation technology without the host pipe. The second 300 ft install was in 8" nominal diameter ductile iron pipe with and without mortar lining simulating the roughness and variability of a degrading pipeline after cleaning.

The Sonotube® trial run produced 8" diameter samples for ASTM 2412-11: Standard Test Method for Determination of External Loading Characteristics of Plastic Pipe by Parallel-Plate Loading and ASTM 2290-09: Standard Test Method for Apparent Hoop Tensile Strength of Plastic or Reinforced Plastic Pipe. These field samples also demonstrated the flexibility of the product with a 30% defection without failure and no plastic deformation as expected from the linear elastic properties (Hindman 2014). This adds additional confidence in the life cycle performance. Unexpected external loading when a pipe is depressurized might cause significant deformation, but such deformation would be reversed by the internal pressure as a pipeline returns to service. In addition to supporting the overall application of CFRP's in pipe, these tests help further validate not only coupon testing but theoretical design calculations. In addition to the element testing, coupons were taken from the host pipe in the longitudinal direction to validate the material properties of the installed pipe which was used to confirm the material design properties.

The ductile iron trial run implemented a testing program of service connection testing with variable protrusion depths and surface preparation methods. Samples were made to test the ability of the rehabilitations technologies' bond strength, under various surface preparation, to prevent leaks at service connection locations. Sixteen samples were tested under a modified ASTM D1598-02 (09): Standard Test Method for Time-to-Failure of Plastic Pipe Under Constant Internal Pressure, demonstrating initial short term leak protection up to 300 psi, the design pressure capacity of the host pipe, for robotically surface prepped ductile iron. This process and culmination of testing provides a significant portfolio of validation for the rehabilitation technology as a fully structural liner that extends beyond the requirements of AWWA M28 as a class IV liner. Although theory serves us well, initial assumptions must be validated with full scale testing. Despite the significant monetary and time investment, full scale testing is crucial to validate the claims produced by manufacturers of developing technologies. Currently, no guidelines/codes have been developed for CFRP renewal and strengthening of small diameter pipelines for the water industry because the developing technologies are so new. Because these technologies are not fully considered in the development of current codes, a significant understanding of long-term material properties validated through physical testing are required to extend the ability for technologies to extend the current codes/guidelines reach for structural applications in addition to driving the development of additional guidelines and codes.

CONCLUSION

As applications in engineering extend beyond the assumptions of codes and guidelines, the assumptions need to be reevaluated and additional considerations must implemented to ensure effective applications with good, sound engineering judgment. There is no replacement for full scale testing. This is well known to everyone on a committee or code council working to better equip those involved with the tools to protect people and our infrastructure. However, it is imperative that we review the base assumptions on which we are building our understanding and adapt accordingly to effectively utilize developing technologies to rehabilitate infrastructure and remove blight.

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Integrated Technology Applications for Effective Utility Infrastructure Asset Management

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Abstract

According to ASCE's "2013 Report Card for America's Infrastructure", the United States received a D+ for an overall "below average" rating. Drinking water and wastewater utility infrastructure both received a D rating. Comprehensive utility asset management is becoming increasingly important as aged and degrading infrastructure begins to fail. Managers of these utility systems constantly face funding and scheduling challenges to meet the demands of the country's growing population. However, effective utility infrastructure asset management does not always have to involve daunting or expensive processes. Integration of engineering technologies – such as geographic information systems (GIS), global positioning system (GPS)-enabled devices, advanced spreadsheet applications, pole-mounted viewer/recorders (pole cameras), and web-based user interfaces, yield a multitude of dynamic data and statistics to end users at the touch of a button during and after infrastructure condition assessments. This paper details a unique approach to infrastructure asset management in which managers of small to medium-sized utilities (generally less than 500 miles of linear assets) can decrease cost and increase productivity through selection and integration of appropriate engineering technologies.

OVERVIEW

Although a myriad of engineering technologies are available to facilitate infrastructure condition assessments and asset management applications for small to medium-sized utilities, the level of detail required is dependent upon the needs and resources of the client. The integrated approach discussed below has been successfully deployed at several Department of Defense military bases, specifically within the United States Air Force (Air Force). Select bases across various commands have required inspection-based condition assessments to support asset management programs established within the Air Force. These condition assessments have also investigated the extent to which aged and degraded utility infrastructure within the bases contributed to environmental compliance issues.

AECOM developed the current infrastructure prioritization model and asset management process for Air Force clients by first conducting a multi-utility pilot project at Joint Base Elmendorf-Richardson, Alaska. The approach was then implemented in subsequent drinking water, wastewater, and stormwater sewer condition assessments at other Air Force bases across the country. The following methodology will focus primarily on wastewater applications. Utilization and integration of four main engineering and data collection technologies – GIS; mobile, GPS-enabled devices; pole cameras; and web-based user interfaces – is essential to the successful execution of this approach. Incorporating these technologies as a comprehensive package ultimately provides a simple, client-sustainable asset management solution once the initial condition assessment has been performed.

METHODOLOGY

Pre-field Data Evaluation. Conducting utility infrastructure condition assessments requires spending time in the field, thoroughly inspecting assets on site. Before deploying field inspection teams, existing system data are combined and connected in a GIS database (geodatabase) from various sources. Examples of data sources include construction drawings, utility personnel knowledge obtained through interviews, computer-aided design (CAD) files, and current information in the geodatabase. Pre-field data evaluation identifies areas of uncertainty or inconsistency with system mapping that can be corrected by technicians prior to or after the field assessment. It can also identify locations requiring detailed, visual inspection during the field assessment. For example, a utility shop technician could identify a section of pipe in a wastewater system where blockages have occurred on numerous occasions. In turn, the pipes in this location would be prioritized for direct inspection with the pole camera to identify a blockage source and obtain a visual of the pipe condition.

Prior to mobilization, comprehensive system maps are developed in GIS that identify inspection requirements for field teams and provide a means of hard-copy documentation for field notes and observations. The field notes function as a redundancy to assessment data collected in the field with mobile, GPS-enabled devices.

Field Inspection and Data Collection. Ruggedized, mobile GPS devices with sub-meter GPS accuracy and user-friendly data collection interfaces and software are vital to accurate field data collection and continual geodatabase updating. With these devices, assets are GPS-located, assessed, and edited in the field. Because condition assessment criteria are different for each asset (i.e. manholes, pipes, and lift stations) in utility infrastructure systems, customized assessment forms developed in GIS are incorporated into the data collection software. Pre-loaded drop down menus created and customized with CartoPac data collection software, are linked with specific asset attribute fields, ultimately reducing the amount of time spent per asset during inspections while minimizing typographical errors. Examples of a GPS device and a mobile data entry form are shown in Figures 1 and 2.

Figure 1. Field GPS Instrument (Leica CS25 GNSS Shown)

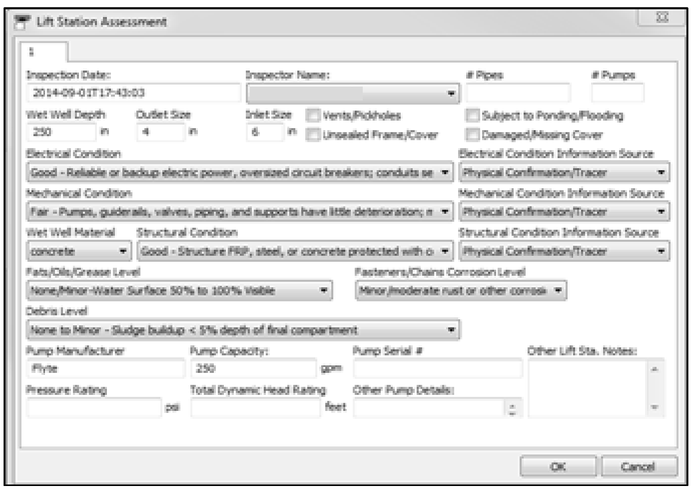


Figure 2. Example Device Data Entry Form

During the field investigation phase of the condition assessment, sections of pipeline are visually inspected using the pole camera according to plans strategically developed in the pre-field data evaluation phase. The specialty device is a basic pole-mounted viewer equipped with a camera perched on a telescopic, pivoting rig with illumination lamps and a remote viewing interface. Sewer manholes are the most direct points of access to pipes across the system and are thus pre-selected in the inspection plan. After the condition of the manhole has been inspected and entered into the GPS device, the pole camera is lowered to pipes so that photographs can be taken as the condition is assessed by a technician operating the remote viewing interface. Attributes such as pipe material, diameter, and orientation are also noted during the inspection to confirm and/or update existing data in the geodatabase. Assessment data can then be entered into the GPS device as the inspection takes place

One of the primary advantages of this approach with respect to pipeline inspection is that not every foot of pipe is directly evaluated with closed-circuit television (CCTV) inspections. First, pole camera locations are strategically selected in areas of uncertainty, where issues have been identified, and/or where pipe material and age (based upon installation date) are statistically correlated with anticipated structural condition. For example, a vitrified clay pipe installed in the 1940s is more likely to be defective and require repair or replacement than a polyvinyl chloride pipe installed within the past decade. Experience using this method at multiple Air Force Bases has shown that direct visual inspection of up to 35 percent of total pipe length within a small to medium size wastewater system is appropriate to gain an overall condition assessment of similar pipes that are not directly inspected. Essentially, the length of pipe and locations selected for pole camera inspection need to be sufficient to extrapolate data when the condition assessment is complete.

Parameters required for the asset management prioritization model are those by which assets are assessed. For example, structural deficiencies in a pipe – such as breaks, cracks, corrosion, or deformation – are noted if observed during the field investigation, along with degree of severity. The pole camera utilized in this approach has a reported range up to 400 feet, although field use reports a practical range up to 100 feet in a straight run of pipeline. Ultimately, effective viewing range of the pole camera can be limited by pipe bends, small diameters, and manhole bench construction hindering ease of access. Sample photographs of the pole camera in use during an inspection and a wastewater system pipe taken with the pole camera are shown in Figures 3 and 4.

Figure 3. A technician operating the pole camera during a pipe inspection

Figure 4. Examples of zoomed-in (left) and wide-angle (right) imagery of a vitrified clay pipe

Data collected in the field is uploaded daily from GPS devices to an online server throughout the duration of the field investigation. The dynamic integration of mobile data collection devices and the server where data is stored is facilitated by an enterprise asset management software solution. This intrinsic, technological relationship enables field teams to track progress of the assessment in real-time and ensures that no asset goes overlooked while on site.

Post-field Data Reconciliation. Upon completion of the field inspection, GPS-updated asset locations and associated condition and attribute data are reviewed for accuracy and completeness. Remaining data gaps with respect to various assets are resolved from field observations, often combined with knowledge ascertained from other sources (i.e. the condition and disposition of surrounding assets). Data gaps can include unknown pipe materials, buildings without service lines, inconsistent flow directions, etc. In some instances, developing engineering assumptions with respect to reconciling data gaps is required. At this point, the process of extrapolating pipe condition data gathered from pole camera inspections can begin.

Data extrapolation is a function of engineering judgment that applies observed conditions of assets to those not observed, where appropriate. A cornerstone of this approach to asset management initially replaces extensive CCTV inspection of up to thousands of feet of pipeline with pre-selected areas for a quick, direct inspection. Therefore, conditions must be applied to the remaining percentage of system pipeline to fulfill requirements of the prioritization model – utilizing pipe installation date and material, at minimum. Essentially, structural condition of pipes observed in the field is applied to those not observed as long as the material and installation year are the same. Where pipe installation dates are not known, assumptions are formulated based upon the material of the pipe and the median of typical historical use. This method is only recommended for small to medium-sized utilities.

Upon completion of the data reconciliation process, each asset is assigned a risk score developed from a risk matrix. As detailed in Table 1, this matrix combines the probability that an asset will fail with the consequences of failure to stakeholders (criticality). For example, an asset with a high probability of failure (i.e. a concrete pipe observed near collapse in the field) and high criticality (i.e. large-capacity

wastewater pipe serving several facilities across multiple basins) is an extreme risk to stakeholders in the event of failure.
Therefore, this pipe yields a numerical score reflecting this extreme risk and is prioritized for rehabilitation in the form of structural repair or replacement.
Repairing a single segment of pipe in isolation is uncommon after a large-scale condition assessment has occurred. Therefore, projects comprised of multiple pipes within the vicinity that also require rehabilitation are recommended for the client.

Table 1. Sample Risk Matrix

Probability of Failure	Consequence of Failure				
	Low	→	Medium	→	High
Low	Negligible	Negligible	Low	Low	Moderate
↓	Negligible	Low	Low	Moderate	Moderate
Medium	Low	Low	Moderate	Moderate	High
↓	Low	Moderate	Moderate	High	Extreme
High	Moderate	Moderate	High	Extreme	Extreme

Limitations. Partial, customized pole camera inspection and post-field condition data extrapolation of an entire system is an effective strategy for small to medium-sized utilities with less than 500 miles of pipeline. However, this approach may not work for larger utilities with hundreds of miles of large diameter pipes (typically greater than 24 inches). In these systems, man inspection or CCTV surveys are most appropriate.

While the pole camera can provide a good overall view of the pipe to determine general condition, it does not provide sufficient information to establish the most suitable rehabilitation method between repair and replacement. Rehabilitation methods recommended to the client are based on aforementioned limitations of the pole camera. This approach to asset management with customized pole camera inspection has the advantage of expediency and functions as a means to identifying areas that require more investigation for potentially problematic assets. These inspections, along with the subsequent extrapolation process, can help direct where more focused CCTV inspection is implemented as opposed to investing a great deal of time, manpower, and financial investment in a comprehensive CCTV survey of the entire system.

For these reasons, thorough engineering analysis should precede each proposed project in order to verify that the recommended solutions are technically appropriate. At a minimum, the engineering analysis must include a CCTV survey of identified pipes in order to verify the extent of conditions observed in during the field inspection.

Client-sustainable Asset Management. Practicality of the prioritization model and ongoing asset management is dependent on the accuracy and efficacy of underlying data in the geodatabase. Future facility modifications and sanitary sewer projects must be simultaneously verified in the field and updated in the geodatabase. Moreover, the client must be equipped with the user-friendly tools necessary to update their asset management system.

The client receives an updated geodatabase, advanced spreadsheet applications that compute risk scores utilized in the prioritization model, and a dynamic, web-based user interface connected to the geodatabase for generating individual asset server reports upon completion of the condition assessment. The spreadsheets, connected to attribute tables within the geodatabase, demonstrate how attributes are weighted and calculated into the prioritization model for each asset. Server reports are generated through reporting service software connected to information also stored within the geodatabase. These server reports then graphically display attribute and condition data for each asset, along with risk scoring data and assessment photos from the inspection. End users can readily access server reports on a webpage without having to open the geodatabase to get information on system assets. Reports are automatically updated when changes to the geodatabase are made. There are multiple types of software that can be configured to work together in this way. AECOM utilized ArcGIS, CartoPac, Microsoft SQL Server Report Builder, and BIRT (Business Intelligence and Reporting Tools). Figure 5 is an example of a sanitary sewer pipe server report.

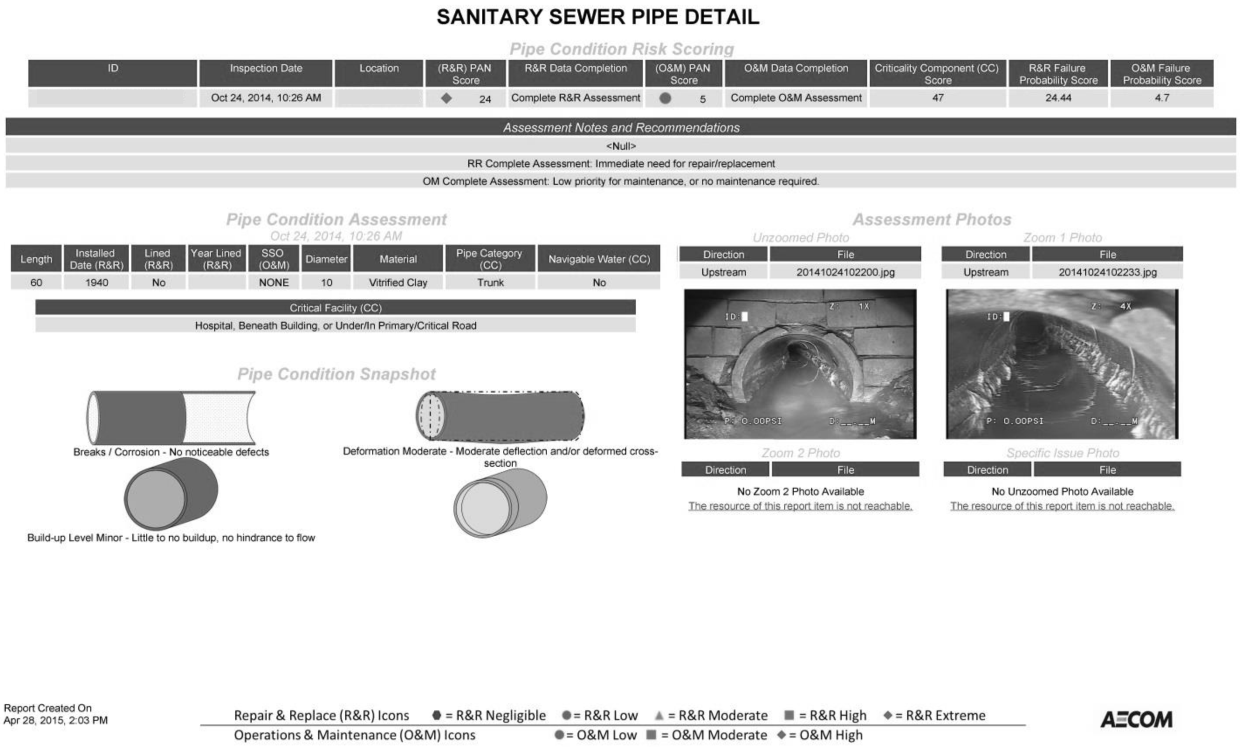
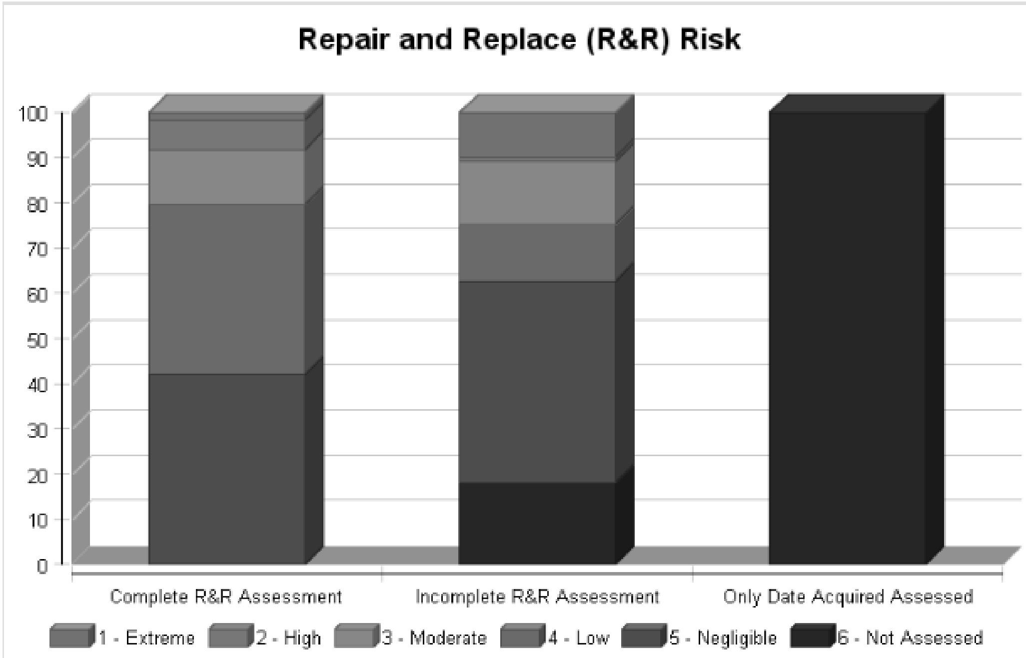


Figure 5. A sanitary sewer asset server report detailing pipe condition and risk data

The prioritization model utilized in this asset management approach is customizable based on client needs. The primary advantage of server reporting is that clients can easily track the status of their most critical assets, especially when developing capital improvement plans and programming and scheduling projects. This proactive style is preferable to a reactive one, where system managers must respond to financial and operational consequences after a failure has already occurred. Figure 6 is a snapshot of a server report generated to track and display assets with the highest risk.



Top Repair and Replacement (R&R) Scores

ID	Risk Level	Risk Description
PIPE 00382	24	RR Complete Assessment: Immediate need for repair/replacement
PIPE 00400	24	RR Complete Assessment: Immediate need for repair/replacement
PIPE 00401	24	RR Complete Assessment: Immediate need for repair/replacement
PIPE 00377	18	RR Complete Assessment: Evaluate repair/replacement alternatives. Repair/replace in the next 2 to 5 years.
PIPE 00381	18	RR Complete Assessment: Evaluate repair/replacement alternatives. Repair/replace in the next 2 to 5 years.
PIPE 00035	18	RR Complete Assessment: Evaluate repair/replacement alternatives. Repair/replace in the next 2 to 5 years.
PIPE 00165	17	RR Complete Assessment: Evaluate repair/replacement alternatives. Repair/replace in the next 2 to 5 years.

Figure 6. Graphical display of assets with corresponding risk scores

CONCLUSION

Experience has proven that the most nimble and client-sustainable asset management systems require technology that is relatively affordable and easily accessible small to medium-sized utilities. Applying integrated technology applications as essential tools before, during, and after comprehensive infrastructure condition assessments results in a robust asset management data collection and organizational methodology. Furthermore, these integrated tools assist clients in developing effective capital improvement along with operation and maintenance strategies long after consultants have completed field investigations. Once an assessment database and risk model are established, they are easily updated by end users – such as utility managers and field technicians – for ongoing system management and technical support. The process is simplified as extensive, paper-based asset reports and costly investigations are eliminated. Integrated asset management technology facilitates a one-stop system, decreases costs, and increases productivity.

Beyond Water Audits into Asset Management: The Process of Non-Revenue Water Reduction and Revenue Enhancement Activities

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Abstract

For many water utilities, the true amount of non-revenue water is a mystery, or at least a guess. With the growing use of tools like the AWWA Free Water Audit Software, many utilities are beginning to have a better understanding of how their water system is operating. But with this realization, comes the next question, which is how to economically reduce water losses. There are many things that can be done, but which should be done, and to what level of implementation are the tough questions. The concept of economic level of losses is important, but most water systems will not be close to determining this level after first assessing their non-revenue water. This paper will go describe the approach to identifying the quantity and components of non-revenue water through an initial audit, bottom-up activities, evaluating, selecting and prioritizing corrective approaches, and implementing water loss control activities for effective results. The ability to change course in mid-stream is also important, and so accurate progress reporting is critical. A comprehensive non-revenue water reduction program integrates and informs with other utility management functions, such as asset management. In particular, the level of rehabilitation and replacement of pipelines is related to the non-revenue water goals and economics. This relationship between asset management and non-revenue water becomes the cornerstone for ongoing planning and improvement.

BACKGROUND

For a very long time, water systems used the antiquated terminology of "Unaccounted-for water" to classify the amount of water lost between production and sales. However, there was no industry standard definition or calculation for this term, which was often expressed as a percentage, and translated nothing about the financial impact of water loss. In order to improve this, the American Water Works Association (AWWA) through Water Loss Control Committee released a committee report in August 2003, which outlined a methodology developed in partnership with an International Water Association (IWA) workgroup. Since that time, the AWWA and water industry has adopted this IWA/AWWA methodology as the best practice, (AWWA website). Essentially, this methodology classifies all water that enters the distribution system into standard categories, so that no water goes "unaccounted-for". This allows for a complete water balance to be performed, as shown in Figure 1. Further explanation of this water balance and methodology is contained in the AWWA M36 Manual, Third Edition, 2009.

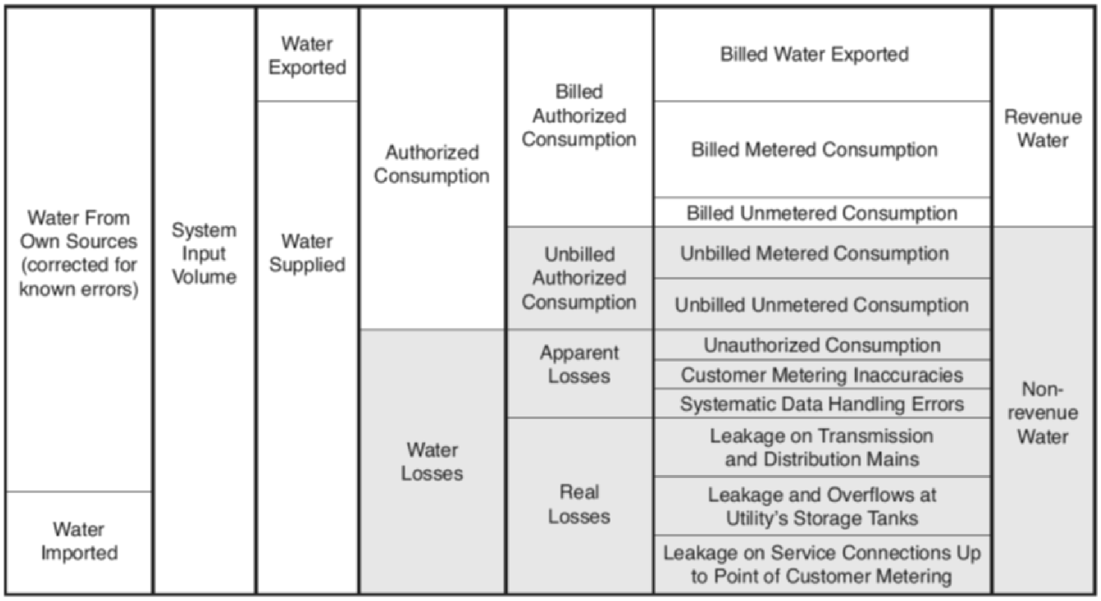

Figure 1. AWWA/IWA Water Balance.

The following are a list of key definitions used throughout this paper:

- **Non-revenue water (NRW)**: the difference between water entering the distribution system and the water billed to users/customers
- **Water loss:** the difference between water entering the distribution system and the water used for authorized purposes; also the sum of real and apparent losses
- **Real loss:** water that is lost due to leaks and breaks in pipes, overflows at tanks, and on service connections up to the point of metering (if applicable)
- **Apparent loss**: water that is lost to unauthorized use (theft), customer metering inaccuracies, and systematic data handling errors

In an attempt to increase adoption of this new methodology throughout the water industry, the AWWA Water Loss Control Committee has developed a Free Water Audit Software © that can be used to perform a top-down assessment of the water balance shown in Figure 1. This water audit software is available for free from AWWA and is used in Microsoft Excel. The software has been tested, revised, and improved since 2005, and version 5.0 was released in 2014. Some key features of the software are that the inputs required for the audit are given a "grade" by the auditor so that the overall "data validity" can be assessed, as well as the calculation of key operational and financial performance indicators.

OBJECTIVES

Objectives of this methodology are to allow water systems to evaluate their non-revenue water with increasing confidence and determine activities that can be implemented to reduce the components of non-revenue water, in an economic fashion.

NON-REVENUE WATER PROCESS

A process can be outlined to incorporate the water audit in a non-revenue water reduction program. This program integrates with data from multiple sources throughout the water system, and connects with operations, asset management, and billing and customer service in the reduction of non-revenue water. Figure 2 shows this NRW Process and is described in the paragraphs below.

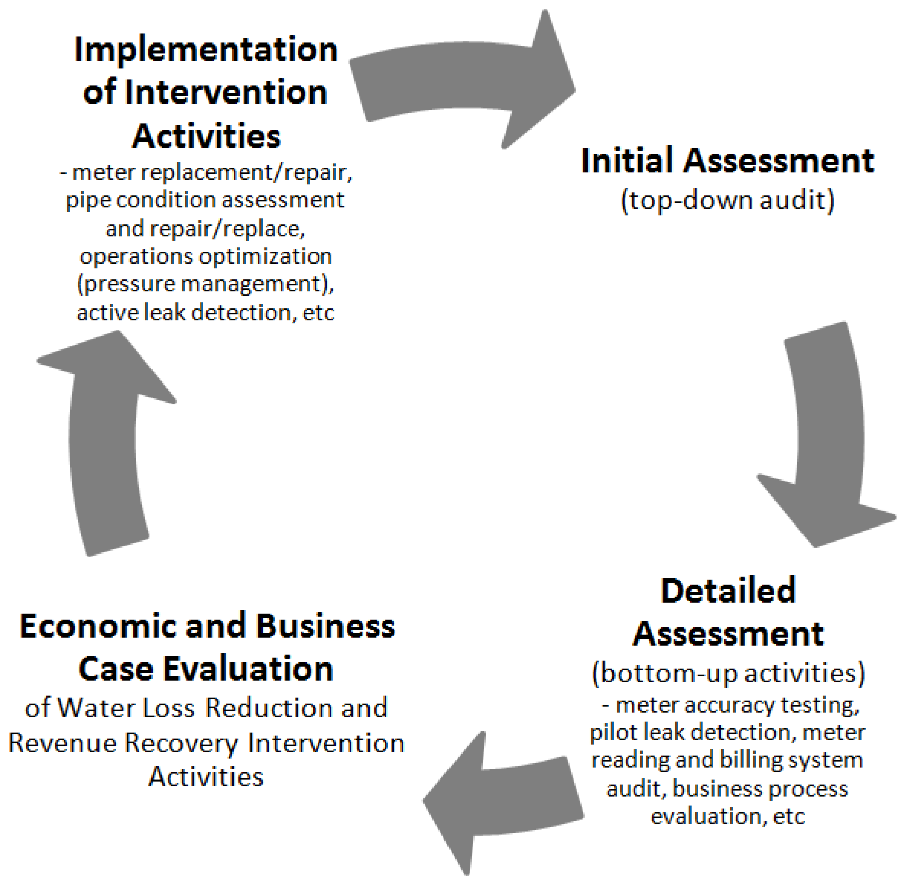

Figure 2. NRW Process.

INITIAL ASSESSMENT

The use of the IWA/AWWA Water Audit Methodology through the Free Water Audit Software allows for water systems to perform a top-down assessment. This is called a top-down exercise because it is primarily meant to be performed using data generally available. When this is combined with the user-entered data grades, an overall data validity is calculated, and the water audit generates the top three inputs that with improvement will improve the data validity.

The top down water audit performed using the Free Water Audit Software calculates some key performance indicators of water loss. These performance indicators can be used by the water system as metrics to track their progress over time, as well as benchmark themselves with other utilities, depending on the indicator. There are financial indicators, as well as operational efficiency indicators.

DETAILED ASSESSMENT

Activities can be performed to improve the confidence in the audit, by increasing confidence in the data inputs, as well as the implementation of best practices. These criteria are outlined in the water audit software, in what is called the grading matrix.

Data grades should be improved to the level at which the overall water audit data validity provides confidence in the use of the results for non-revenue water reduction activities. This includes field verification of production meter accuracy, customer meter testing, evaluation and auditing of billing system practices, and component analysis of real losses.

One of the most important inputs to the water audit is the water supplied. Therefore, the first thing a water system should do is consider their confidence in the accuracy of this number. Activities to verify this number could include using a temporary, secondary meter to verify the accuracy of the primary flow meter or meters for finished water or testing of bulk water import meters. An example of this is shown in Figure 3 below, using a temporary insertion meter to verify the accuracy of the existing flow meter.

Figure 3. Finished Water Meter Testing using a Temporary Insertion Flow Meter.

Other important activities during the detailed assessment process include customer meter testing, assessment of meter reading and billing practices, as well as component analysis of real losses. The first of these is customer meter testing, which is important to understand the rate at which metering accuracy may be deteriorating. For instance, most customer meters in use by water systems are mechanical in nature, and therefore, wear out over time, resulting in reduction of accuracy, especially at low flowrates. A program of testing statistically significant number of meters can help a water system determine their overall customer meter accuracy with a greater degree of confidence. Figure 4 below shows an example of residential-type meter testing. Figure 5 shows an example of the results of customer meter testing.

Figure 4. Customer Meter Testing.

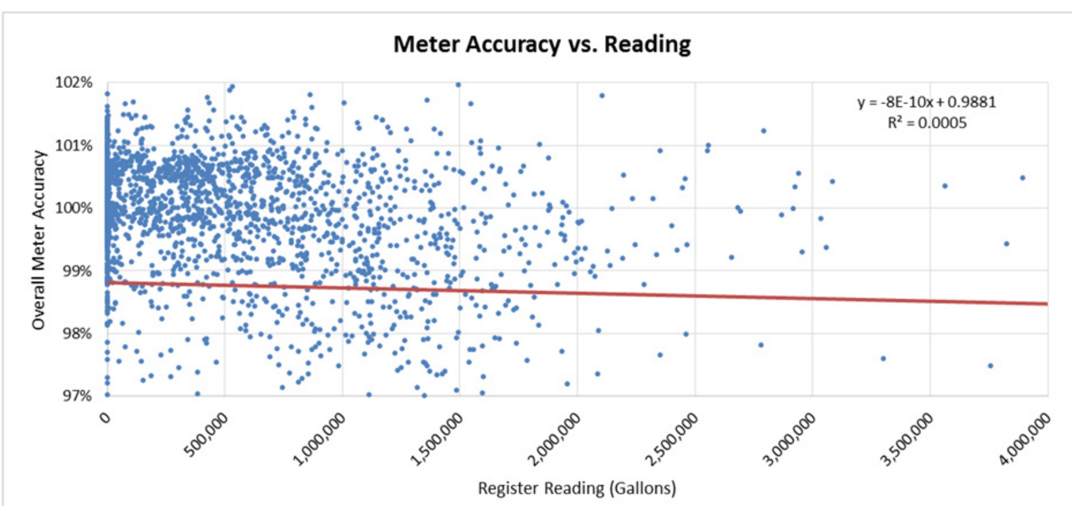

Figure 5. Customer Meter Test Results.

Another practice that can be performed to increase the detail and confidence of the non-revenue water balance is an audit of the meter reading and billing practices, as well as review of the billing system exceptions reporting. This can be done by simply creating a flow chart for the process of a meter reading being collected, recorded, converted into a consumption volume, and a customer bill being created. This can help create more accurate assessments of apparent losses experienced by a water system.

For many water systems, real losses are a greater component, on a volume basis, so it may be worthwhile to understand the components of the real losses. These include reported leakage, unreported leakage and background leakage. By determining how much each of these contributes to the overall real losses, appropriate reduction techniques can be identified. For example, reported leakage and unreported leakage are essentially the same, except that while the reported leakage usually is visible on the surface of the ground and is thus reported and repaired, the unreported leakage does not surface, and may remain leaking for much longer before it surfaces. Unreported leakage is detectable using current acoustic leak detection technology. Background leakage, however, is much smaller in individual locations, and thus is not detectable by these means, but it may still make up a large volume component of the real losses experienced by a water system. Figure 6 below shows a breakdown of these types of real losses that occur in a water system. (WaterRF, 2014)

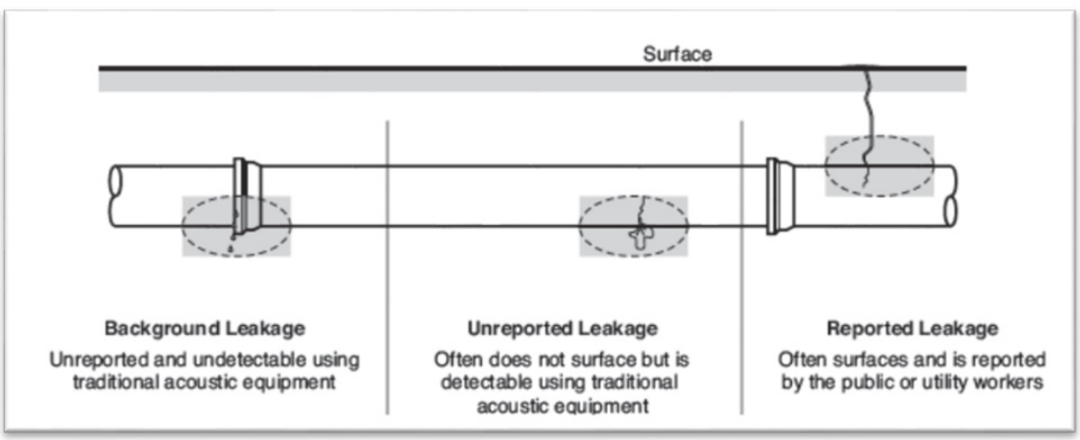

Figure 6. Components of Real Losses.

ECONOMIC LEVEL OF LOSSES

It is important to define the economic level of losses for the purpose of understanding how water loss affects a water system's operation and financial health. The economic level of loss can be applied to apparent losses, or real losses separately.

The economic level of losses can be determined by the cost of various water loss reduction activities and the value of the water loss that is recovered. Those activities that create a positive return on investment can be defined as economically justifiable.

This requires the accurate determination of the value of the components of non-revenue water. These can be determined as shown below:

- Real Loss
  - valued at wholesale rate or variable production cost (usually)
  - Typically \$0.14 - \$1.50 per 1,000 gallons
  - Could use retail rate in water shortage conditions
- Apparent Loss
  - valued at retail rate or customer retail unit cost
  - Typically \$2.00 - \$5.00 per 1,000 gallons
- Unbilled Authorized is based on the type of use
  - Valued at variable production cost (in water audit software)
  - May be political reasons that prevent recovery of some unbilled authorized use

It should be noted, that in general, the total volume of the real losses is typically higher than apparent losses, but the total value of the apparent losses is typically higher because the unit rates applied are different. This illustrates how the different components of non-revenue water impact a water system operations in different ways, as well as the types of water loss control activities each water system may decide to prioritize.

ECONOMIC AND BUSINESS CASE EVALUATION

Based on the value of the recovered water losses, payback calculations can be performed to determine projects to be performed that can reduce water loss. Depending on the components of water loss, various intervention activities can be performed as shown below:

Real loss reduction:

- Active leak detection to reduce unreported leakage
- Improving the speed and quality of repairs to reduce reported leakage
- Pressure management to reduce background and unreported leakage
- Pipeline rehabilitation/replacement to reduce unreported leakage

Apparent loss reduction:

- Customer meter replacement to improve accuracy
- Automatic meter reading/advanced meter infrastructure to eliminate errors in the meter reading process
- Business process evaluation including interaction with the following:
  - Meter reading system evaluation
  - Billing system audit/upgrades
  - Customer service interface and access

For each activity, the economic implementation level can be determined. Using the principles in this paper, water systems can economically justify their investments in water loss control. In the case of apparent loss reduction, increased revenues can be experienced, which is why many water systems choose to perform activities related to apparent water loss control before real losses.

INTEGRATION WITH ASSET MANAGEMENT

Many of the activities listed above are integrated with other components of water system operation and maintenance activities. In particular, pipeline rehabilitation/replacement is directly related to asset management practices. If a water system has a detailed history of water line failures, breaks and bursts, the data can be analyzed to prioritize portions of a water system to be surveyed for leaks, relined and rehabilitated, or replaced and renewed. Understanding the benefits to water loss reduction through these activities can help an asset management program assign further priority to these projects so they are implemented sooner.

IMPLEMENTATION AND MEASUJREMENT OF PROGRESS

As programs are implemented to reduce the various components of non-revenue water, the water system should track and report on progress to make modifications, as needed. This is in agreement with the process shown in Figure 2.

One example of tracking implementation progress can be illustrated by tracking the miles of pipeline that are surveyed using leak detection equipment. This does not necessarily guarantee reduction of real losses, but should provide reductions. The economic goal of miles per year can be used as a metric of progress. If the economic level of active leak detection was calculated, this can become the target length of pipes to be surveyed annually.

In addition, it is important to perform the water audit on a regular periodic basis. This could be annually, using calendar or fiscal or water year 12-month basis. It is not generally recommended to perform auditing on a 30-day basis, due to the difficulty in reconciling the customer meters with the source meter data. However, the use of 12-month rolling totals on a monthly basis has been useful for some utilities to identify trends and issues. Figure 7 below illustrates the implementation of a 12-month rolling calculation of water loss.

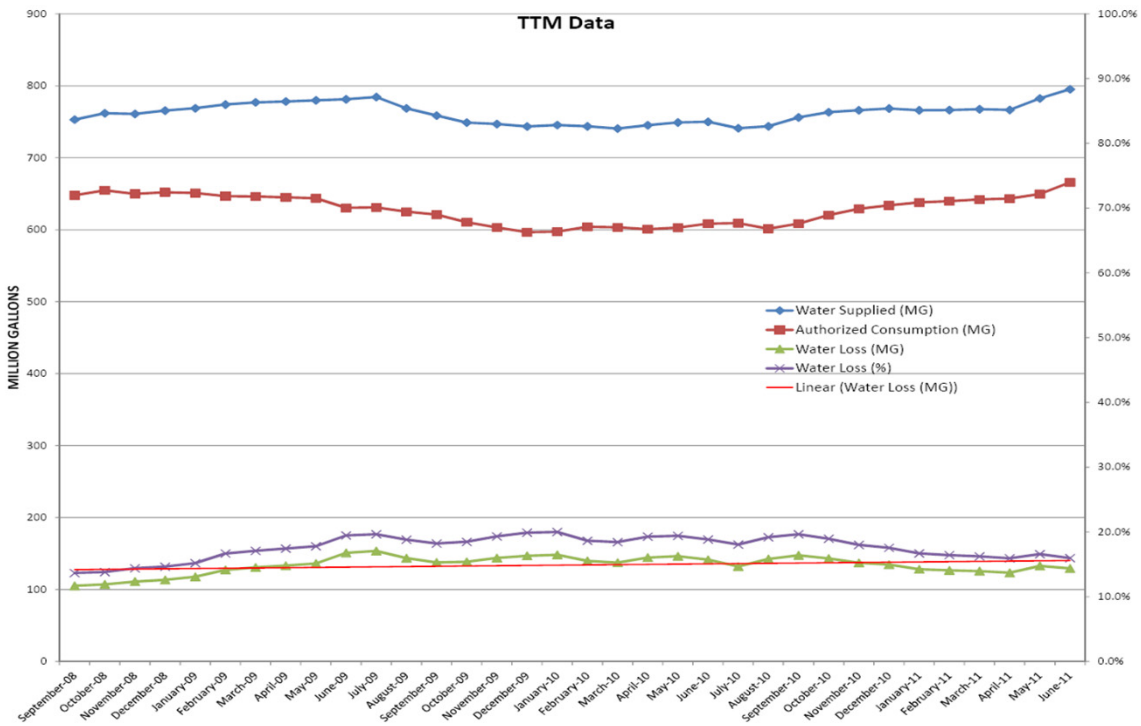

Figure 7. 12-month Rolling Totals of Non-Revenue Water

Finally, the NRW process shown in Figure 2 should be repeated and progress of improvements on the performance metrics should be reported. Periods of abnormal or extreme events may interrupt progress or disrupt calculations, however, and these should be noted. Examples of this may be large customers reducing their use, or extreme weather that may impact customer use, or flooding and soil shifting. A catastrophic failure of a large diameter pipe can also create anomalies.

CONCLUSION

Water systems that implement a comprehensive non-revenue water (NRW) assessment and control program can benefit from understanding the components, and the economic impact to their operations. By improving the confidence in their assessment, they can have greater confidence in the reduction practices they implement. In fact, the economics of NRW reduction practices can be calculated by determining the return on the investment and prioritizing those that have a positive value. It is important to periodically track progress, and report to key stakeholders and leaders. Finally, the process of NRW reduction is one that takes time, and careful selection of intervention activities. However, it can and should be integrated into asset management activities within a water system, to help achieve goals of operational excellence.

In addition, the AWWA Water Loss Control Committee has collected water audits from systems that have volunteered to share their results. These results have been validated, and are published on the AWWA website for informational purposes (AWWA website).

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Fully Structural Renewal of 39-inch PCCP Water Transmission Main with Swagelining™ and HDPE

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Abstract

Gulf Coast Water Authority (GCWA) and the City of League City, Texas were faced with the need to renew a critical 39-inch PCCP water transmission main. The 39-inch PCCP water main, owned by GCWA and operated by League City, required replacement of approximately 6,800 feet along Calder Road. This paper will outline the design phase and selection process as traditional slip lining was considered including an alternate for a compressive tight fitting HDPE pipe. The design criteria required a fully structural solution capable of 125 psi operating pressure. The Swagelining™ process was selected over slip lining due to the additional flow capacity and the contract was awarded to Murphy Pipeline Contractors. This paper will also outline the construction phase, including the Swagelining™ process, the pipe installations performed by Murphy Pipelines, and the challenges associated with the installation of a 1000 mm (39.4 inch OD) DR 17 HDPE pipe. The 2.32-inch wall thickness pipe was pulled through a single swage die in four pulls ranging from 1,250 feet to 2,100 feet in length. The Calder Road Project represents the largest diameter, fully structural pipe installed to date in North America utilizing the Swagelining™ technology. The utilization of this technology with HDPE pipe allowed the owner to meet all design parameters and increase the flow capacity. Swagelining™ offers a solution for pressure pipe renewal that is unique in today's trenchless pressure pipe market as it meets both internal and external loading requirements.

PROJECT BACKGROUND

GCWA had identified a major potable water supply main in need of replacement. The 39-inch PCCP water transmission main, owned by GCWA and operated by the City of League City, feeds a water plant supplying water to the western quadrant of the City of League City. The transmission main, originally constructed in 1971 to supply surface water to the City of Galveston, interconnects between GCWA's Thomas S. Mackey Water Treatment Plant and the City of Houston's Southeast Water Purification Plant. In addition, this approximately 6,800 foot section of Calder Road was scheduled for reconstruction and widening.

DESIGN PHASE AND SELECTION PROCESS

GCWA issued a very complex Request for Competitive Sealed Proposals (RFCSP) to allow GCWA to select the best materials and method for rehabilitation. The proposal included a base price for slip lining allowing HDPE or PVC. Alternate 1 included slip lining with higher pressure classes of HDPE or PVC, and alternate 2 included the Swagelining™ method with HDPE. Although not everything allowed was proposed, GCWA received proposals from five contractors.

After reviewing the cost and capacity of the pipeline, GCWA along with their partner the City of League City awarded the highest grade to Swagelining™ with HDPE based on the evaluation criteria. Final Internal Diameter (ID) with Swagelining™ resulted in 33.86-inches, over 4-inches larger than slip lining with PVC or HDPE (Figure 1). The City of League City could satisfy their demand with the slip lining option; however, since the Calder Road potable water transmission line also provides an interconnect between the City of Houston's Southeast Water Purification Plant and the GCWA's Thomas S. Mackey Water Treatment Plant, GCWA provided funding to pay the difference in cost between slip lining and Swagelining™.

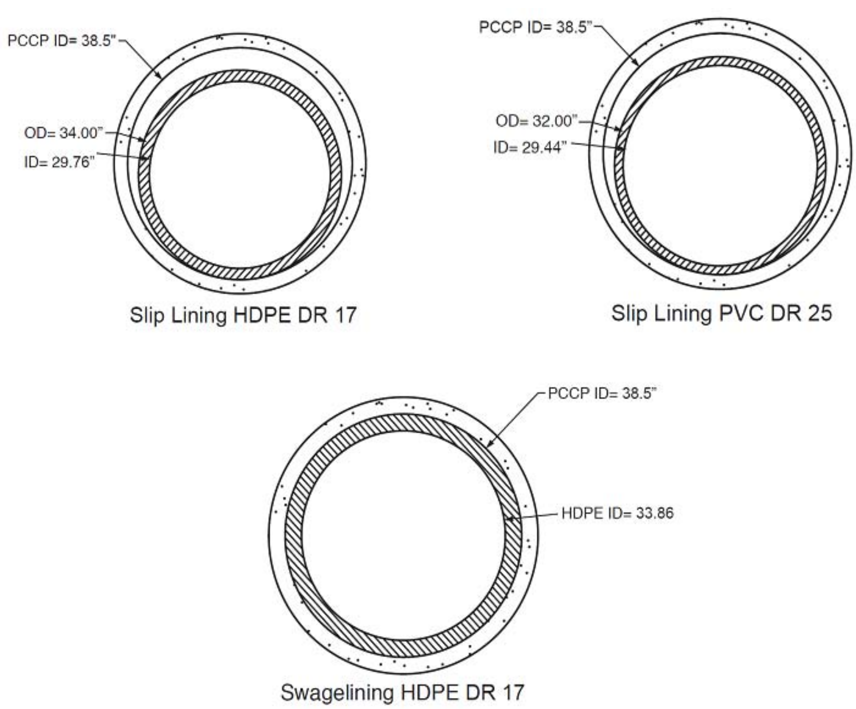

Figure 1: Final ID comparison between slip lining and Swagelining™.

SWAGELINING™ HISTORY AND OVERVIEW

The Swagelining™ technology was developed over 30 years ago by British Gas in conjunction with United Utilities. With an extensive list of successfully completed projects across the globe, the technology has been proven in many extreme projects spanning three decades onshore and subsea. Projects have been completed for water, sewer force main, mining, hydrocarbons, chemicals, bulk products and gas distribution. The overall confidence of the technology originates from an extensive physical testing program conducted by British Gas over several years. The process was established after extensive analysis of material behavior during and after die reduction. A major result of the research and development program was the development of the liner system design software. This software program, which is utilized for each project, ensures installation stresses do not compromise the integrity of the HDPE.

The Swagelining™ technology specifies a PE4710 High Density Polyethylene (HDPE) pipe with an outside diameter larger in size than the inside of the host pipe to be renewed. After the HDPE is butt fused to correspond to the pull distance, the pipe is pulled through a single reduction die immediately before entering the host pipe. This reduces the HDPE pipe temporarily below the ID of the host pipe allowing it to be inserted (Figure 2). While the towing load keeps the HDPE under tension during the pull, the pipe remains in its reduced size. The HDPE remains fully elastic throughout the reduction and installation process. As the liner pipe is not permanently deformed by Swagelining™, the release of the towing load after insertion is the catalyst for the liner to revert back towards its original size. As its original size is larger than that of the host pipe, the HDPE pipe expands until it is halted by the inside diameter of the host pipe. This produces a residual strain that is locked in the liner and maintains pressure against the inside of the host pipe, even in the absence of internal pressure from the product conveyed.

Figure 2: Swagelining™ process as HDPE is pulled through the reduction die.

The effectively natural compressive tight fit produced by Swagelining™ provides value for clients looking to maximize the final ID of the pipeline. Due to the tight fit, thin walled HDPE liners and semi-structural HDPE pipe can be installed in which operating pressure is delivered through the host pipe. In circumstances such as the Calder Road 39-inch PCCP water transmission main which required a fully structural stand-alone solution, Swagelining™ can install a fully structural HDPE PE4710 pipe such as DR 17 with a working pressure rating of 125 psi, allowable total pressure during recurring surge events of 187 psi and allowable total pressure during occasional surge events of 250 psi (ASTM F714, ASTM D3035 and AWWA C901). Higher working pressure ratings above 125 psi can also be achieved. In addition to meeting internal pressure loads, the HDPE installed met all external loading requirements.

CONSTRUCTION PHASE

Calder Road runs parallel to 1-45 in League City, Texas. The urban area is a combination of residential and commercial use, with a Big League Dreams Sports Park in the middle of the project. The project limits encompassed a very tight area as Calder Road is a single lane two way road in which the allowed work area was no wider than 25 feet. The 39-inch PCCP water transmission main was located along the edge of the pavement among a congested utility corridor. While shutting down both lanes would have eased construction, only a one lane shutdown was allowed.

The rehabilitation of the 6,800 linear feet of the PCCP 39-inch diameter waterline included the replacement of four 36-inch diameter butterfly valves, additional three 36-inch x 24-inch diameter tees with gate valves, the replacement of two air relief valves, the addition of three blow-off valves, installation of a 39-inch x 36-inch flanged reducer, placement of two large thrust restraint blocks, and a bypass (Figure 3, Figure 4 and Figure 5). The placement of valves, blow-offs and tees were relocated based on the constructability of the project and the needs for the future expansion of the City of League City Water Plant on Calder Road. The north end of the project was extended about 300 linear feet to an existing 36-inch diameter butterfly valve. The south end of the project required the addition of a flanged reducer, butterfly valve and blow-off. Thrust restraints were required at the north and south end of the project to protect the existing transmission main from stresses during construction. Each thrust restraint included an ellison type pipe clamp along with over 40 cubic yards of concrete. During construction the existing water plant was connected to the City of Houston water supply or the GCWA water supply through a 12-inch diameter bypass laid mostly above ground. The project layout was designed to meet the needs of the roadway construction along with the expansion of the Calder Road Water Plant.

Figure 3: Left and middle picture of 24-inch stub out by sidewall fusion with Flange Adapter connected by downhole butt fusion. Right picture of 24-inch Gate Valve bolted on to Flange Adapter.

Figure 4: Left picture of downhole butt fusion connecting HDPE in receiving pit. Middle picture of 36-inch side actuated butterfly valve. Right picture of 39-inch by 36-inch PCCP flange reducer.

Figure 5: HDPE connections using Flange Adapters with stainless steel back up ring and stainless steel bolts.

To minimize the impact of the project to the surrounding community Murphy Pipelines designed the project layout in which four installations ranging from 1,250 feet to 2,100 feet in length were accomplished. The long pull lengths were beneficial as they allowed for long fused sections of HDPE to be installed eliminating future leak potential and aided in the reduction of excavations by 87% of what open trench would have required.

For each pull, the 50 foot lengths of HDPE were butt fused using a rolling McElroy 1648 machine to correspond to each pull length. After each fusion weld cooled, the external roll-back bead was removed to allow clearance through the swag die. While the pipe was fused, crews performed a visual inspection of the interior of the PCCP waterline. This step is critical as it identifies any major obstructions, location and degree of bends and condition of host pipe interior which determines if any cleaning is required. Finally, a proving pig was pulled through. A proving pig is a short section of HDPE fabricated one to two millimeters larger than the installation OD of the HDPE during Swagelining™ operations. Its purpose is to eliminate risk by ensuring a free bore path.

Once a free bore path is confirmed, Swagelining™ operations would begin (Figure 6). To complete each pull, a specific bank shoring plan was implemented to compensate for the amount of force required to pull the long lengths of HDPE with a wall thickness of 2.32-inches through a single swage die. Two types of constant tension pulling equipment were used for the project; Hammerhead 173 ton pulling machine and a TT Technologies 143 ton pulling machine. Both machines performed well and without incident. As part of the liner system design process, Murphy Pipelines utilized their software program to ensure installation stresses on the HDPE met the ASTM standard for the tensile yield design factor.

Figure 6: 1000mm (39.4-inch) OD HDPE pipe enters single swage die. The HDPE pipe is temporarily reduced below the ID of the 39-inch PCCP host pipe to allow for insertion.

After the HDPE pipe was completely pulled through the host pipe (with pull lengths of 1,250 to 2,100 feet), the pulling force was removed. This allowed the HDPE to naturally revert back towards its original diameter until halted by the inside diameter of the host pipe forming a compressive tight fit (Figure 7). While dependent on ambient temperatures, the HDPE is typically allowed to relax overnight to regain full reversion for most thin walled and semi-structural Swagelining applications. Due to the thicker wall of this fully structural application, the HDPE needed longer to revert and regain full reversion.

Figure 7: Tight compressive fit of HDPE after reversion.

SUCCESSES AND CHALLENGES

A major challenge with the project was the location of working within a tight utility corridor with limited room for construction activity. This challenge was addressed early on through extensive communication with all parties involved with an emphasis in working with local businesses and homeowners to understand and meet their demands. The success of this project ultimately required an extensive amount of team work and coordination. GCWA, League City, ARKK Engineers and a number of other local agencies showed great resolve in working with Murphy Pipeline crews to properly plan, adapt and execute the project.

The other major challenge was installing a fully structural HDPE PE4710 DR 17 pipe with a 2.32-inch wall thickness. The thick walled HDPE pipe required more tonnages during installation than most thin walled or semi-structural Swagelining™ applications. This placed a higher importance on bank shoring, HDPE butt fusion operations and pulling equipment.

CONCLUSION

The Calder Road Project represents the largest diameter, fully structural pipe installed to date in North America utilizing the Swagelining™ technology. The utilization of this technology with HDPE pipe allowed the owner to meet all design parameters and increase flow and capacity. The larger final diameter with Swagelining™ vs. slip lining had significant benefits for the project economics. Gulf Coast Water Authority will be able to realize a higher value by delivering more water to its customers, both now and in the future.

As communities across North America face the challenges of aging medium and large diameter water transmission and sewer force mains, Swagelining™ has been proven as a technology that can add remarkable value for renewal and replacement. The method's advanced engineering agenda through research and development coupled with its ability to meet various internal pressure requirements from thin walled to fully structural, including designing for external loading make Swagelining™ a vital method to be considered.

City of Baltimore SW Diversion 78-in. Diameter PCCP: 2,140 LF Continuous Carbon Fiber Pipe Rehabilitation

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Abstract

In 2009 a CCTV/sonar inspection revealed that many areas of Baltimore's Southwest Diversion Sewer (SWDS) were in need of repair. Following this inspection, the City of Baltimore divided the Southwest Diversion sewer repairs into phases for the purpose of retaining consulting engineers and prioritizing repairs. For the Phase III repairs, the City and their consulting engineer, Rummel, Klepper & Kahl (RK&K), worked for several years on project planning and analysis to determine the most effective options to repair the RCP gravity and PCCP pressure segments of this major sanitary sewer conveyance system. A Cured-In-Place–Pipe (CIPP) repair system was selected for the gravity portions and Carbon Fiber Reinforced Polymer (CFRP) for the repair of the pressure portions because each system could provide the necessary repairs with minimal loss in the system's hydraulic capacity. Carbon fiber systems have been typically used by water and wastewater pipeline owners across the United States to provide full structural repairs to distressed and damaged pipeline segments. These specialized materials are designed to meet a variety of installation conditions, including stand-alone repairs where all internal pressures and external loads are assumed by the CFRP without relying on the existing structure for strength. The CFRP scope of work for the SWDS Rehabilitation Project represents the largest continuous installation of carbon fiber completed to date for an internal repair of a large diameter pipeline in the United States. The project required extensive planning, design and an expert installation crew trained and certified for installation of carbon fiber materials. In addition, a comprehensive inspection and QA/QC process was

used throughout the installation process to insure the long-term success of the repair. This paper will address the successes and challenges of the SWDS Rehabilitation Project throughout all stages of the project, including options analysis, design, bidding, best installation practices, collaborative design approaches, coordinating large-scale trenchless repair system installations and inspection.

PROJECT BACKGROUND AND OPTIONS ANALYSIS

The City of Baltimore DPW processes wastewater for approximately 1.6 million residents of the metropolitan area at two (2) wastewater treatment plants (WWTP). The SWDS transports 25 percent of the wastewater generated within the Baltimore metropolitan area, including portions of Baltimore County, Howard County, and Anne Arundel County, to the Patapsco Wastewater Treatment Plant for processing. The City collects and treats upwards of 250 million gallons of wastewater daily and maintains over 1,400 miles of sanitary sewer mains. As part of the system, the SWDS is approximately 7.9 miles in length and is critical to the sanitary collection operations.

The SWDS is comprised of Class III and IV reinforced concrete pipe (RCP) in the gravity sections and Class IV pre-stressed concrete cylinder pipeline (PCCP) in the pressurized sections that range in size from 78- to 102-inches in diameter. In 2009, deficiencies were identified in both the gravity and pressure portions of the system. To address these deficiencies, the Southwest Diversion Phase III Rehabilitation project was implemented and included installation of 2,550 LF of CIPP to rehabilitate the 78-inch RCP gravity sections and approximately 2,140 LF of CFRP installation to rehabilitate the 78-inch PCCP pressure portions of the conveyance system. To complete the rehabilitation, sanitary flow in the 78-inch diameter pipeline had to be by-passed. The by-pass system was sized to handle a peak 2-year, 24-hour flow of 152 MGD. Altogether, the project involved temporary by-pass and rehabilitation of approximately 4,690 LF of pipeline.

Prior to selecting the CIPP and CFRP lining systems, the City and RK&K worked for several years on project planning and options analysis to determine the most effective options to repair the City’s major conveyance system. Rehabilitation options were required to address the structural deficiencies in the RCP and PCCP sewers without reducing the hydraulic capacity of the system. The primary rehabilitation methods considered included CIPP lining, CFRP lining, slip-lining and replacement. CIPP and CFRP were selected for use on this project because they minimized the diameter reduction (primary reason slip-lining was not selected) and also minimized the disruption to the surrounding residential communities and intersecting streets (primary reason replacement was not selected). For the gravity portions of the pipeline, CIPP was the selected rehabilitation approach, whereas CFRP was deemed the most feasible solution for the pressurized region of the pipeline.

SPECIFICATION DEVELOPMENT AND BIDDING PROCESS

Because of the proximity of repairs and by-pass requirements, the CIPP lining, by-pass system installation, and the CFRP lining were bid as part of the same contract. The City of Baltimore and RK&K worked closely together to prepare detailed specifications for each scope of work. In order to establish minimum competency for potential bidders, the City of Baltimore required that potential bidders or their subcontractors be prequalified by the City in the following prequalification categories: sanitary sewer by-pass installation, carbon fiber lining installation, and CIPP installation.

In addition to the use of prequalification categories, the CFRP lining specification included additional experience requirements for the CFRP lining installer, CFRP design engineer, and the CFRP material manufacturer. To ensure appropriate material pedigree, extensive durability data and material testing as well as an International Code Council (ICC) Engineering Service Report number were required for the epoxy and carbon fiber materials. Because there can be significant variability in performance between different carbon fiber and epoxy materials, the City utilized performance based specification requirements for the design section of the CFRP lining specification. Minimum material properties were listed in the specification as well as the structural demands and minimum safety factors that were required to be accounted for in the design. To ensure design criteria were met, stamped drawings and calculations, developed by the CFRP designer, were required with the bid submission. The drawings and calculations were verified by the City and their consulting engineer, to confirm compliance with the specification requirements.

The SWDS rehabilitation project was competitively bid in 2013 and the lowest responsive bidder was selected. Because of complexities associated with installation and maintenance of a by-pass system through downtown Baltimore City, the costs associated with the by-pass system governed the overall project cost. The successful bid team, Spiniello and STRUCTURAL, coordinated closely to accelerate the construction schedule, minimizing by-pass and overall project costs.

BYPASS SYSTEM

One of the most challenging aspects of the project was that wastewater flows would need to be diverted for the duration of the repairs using a by-pass/flow control system. With peak flows around 152 MGD, the bypass pumping and conveyance system was extensive and required several carefully planned right-of-entry agreements for routing the eight (8) 24-inch diameter high-density polyethylene (HDPE) bypass pipes in order to minimize disruption to the urban areas in which the project scope was located. The HDPE pipe sections were fused on site to create a continuous and leak-free by-pass system for the project as shown in Figure 1. Flows were intercepted upstream of the repairs, pumped through a series of fused HDPE by-pass pipes, and returned to the SWDS downstream of the repairs.

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Figure 1. By-pass system installed for the SWDS Project

The by-pass pipes were configured to maintain site, vehicular and pedestrian access through the project by-pass corridor to minimize disruption to the area. This required right-of-entry agreements with several private commercial property owners. Because of the high cost of bypassing the flows, completing the project within the scheduled pipeline shutdown was critical to the success of the project.

PROJECT COORDINATION EFFORTS

Prior to arriving onsite, extensive collaboration took place between all parties to ensure that the technical and operational details were best tailored to the project needs. Open communication in the form of numerous conference calls as well as several in person meetings was critical to making this process move smoothly and effectively.

Once the project specific technical package was finalized, the CFRP materials manufacturer, STRUCTURAL TECHNOLOGIES, hosted a half-day training session for the City of Baltimore and their engineer. Inspectors, as well as project engineering and project management team members received training in the project specific QA/QC program that was to be implemented on the project. This training session allowed all team members to get technical questions answered as well as make sure all parties were aligned in their expectations regarding the specific logistics of what QA/QC information was to be documented and by whom. In addition, confined space training as well as an overview of the project specific safety program to be implemented on site was covered during this training session. The safety training portion of the program was such a success that City of Baltimore conveyed an interest in incorporating some of the project's best practices into the City's overall safety program.

As a part of the design and construction coordination, it was determined that the CIPP lining repair would be installed before the CFRP system. To provide redundancy in the isolation of the pipeline, a "turnback" from the CIPP liner was left in place after installation and helped to serve as an additional form of isolation redundancy in the pipeline requiring entry for the installation of the CFRP system. This allowed for maximum protection for manned entry to the pipe during the CFRP repairs. Once the CIPP system was completed, a pressurized bladder was also implemented at the end of the CIPP repair to provide additional isolation to the pressure sewer region of SWDS, which was to be lined with CFRP.

Implementation of the CIPP and CFRP lining repair systems required extensive planning to coordinate traffic control, dewatering efforts, bypass system operations, and the sequence through which the CIPP and CFRP systems would be installed. Based on the location of the intermediate access manhole within a 4-lane road, work was able to be isolated so only one lane of traffic in one direction was shut down, allowing for traffic to be kept open in the other lane throughout the duration of the project. The City and the construction team worked together to provide the appropriate traffic control in the affected area. The intermediate access manhole helped to facilitate improved construction schedules for the CFRP scope of work. Due to the close proximity of residents, noise levels for equipment as well as working hours were closely coordinated for this project.

The number of access points and their locations, with respect to the required repairs, can have a significant impact on CFRP installations. Three (3) manholes were provided for the 2,140 LF repair section, one at each end of the repair and one intermediary manhole. These access locations allowed for personnel to minimize the distances traveled for transporting rolls of saturated CFRP to the work location. The intermediary manhole helped to facilitate improved construction schedules for the CFRP scope of work.

Because the CFRP lining process was one of the last steps in work associated with the project, significant focus was placed on construction timing and ensuring that the work stayed on schedule. Detailed project status emails were distributed on a weekly basis, at a minimum, to all key team members to facilitate smooth communication and make sure that any items requiring action could be addressed in a timely manner. The close coordination between the entire project team allowed for any necessary actions to be addressed quickly and contributed to the CFRP lining work finishing two (2) weeks ahead of schedule.

QUALITY CONTROL PROGRAM FOR THE CFRP LINING SYSTEM

Overall quality of the CFRP lining system is determined by conservatism of design, durability of materials installed, experience of workers and supervisors involved in installation, and a thorough QA/QC program. The installation process includes surface preparation, mixing of epoxy, saturation of CFRP layers, and installation of

CFRP and curing. Inspection throughout these processes – including at completion – helped insure a quality CFRP installation.

QA/QC forms were created for each of the 114 segments of PCCP repaired. In order to maintain locations of each pipe segment throughout the repair process, a measuring wheel was used to measure the distances between manholes to the pipe segment. The QA/QC forms provided documentation for the entire CFRP lining installation process. It included pre-construction condition, date of installation for each stage of the CFRP process, unique field conditions, environmental conditions, calibration of the mechanical saturation equipment used to saturate the composite materials, inspection/verification of each layer of material installed, and lot numbers of materials used.

One critical verification which takes place is confirmation that the material properties of the CFRP system applied in the field are in-line with the properties utilized in design. To do this, 12-inch by 12-inch test panels were made utilizing the carbon fiber fabric, epoxy, and saturation equipment from the production runs for the field-installed CFRP lining system. These panels were then cured in the pipe to ensure the same curing environment. After curing, these panels were collected by the Engineer and sent to a certified laboratory to be tensile tested in accordance with ASTM D3039 to gather material properties.

Figure 2. Witness panels utilized to confirm material design values.

As a part of the quality assurance program, inspection "hold points" for each construction region were provided throughout the construction process. The inspection hold points included:

- Verification of surface preparation via bond testing in accordance with ASTM D4541 pull tests (Figure 6)
- CFRP post-installation inspection (Figure 3)
- Top coat post-installation inspection (Figure 4)

Figure 3. Inspection of the Installed CFRP Lining, prior to Top Coat Installation

Figure 4. Inspection of top coat

The QA/QC forms for each segment of PCCP repaired provided documentation for the inspection hold points, installation of the CFRP system, and served as permanent records of the project.

SURFACE PREPARATION FOR THE CFRP LINING SYSTEM

Prior to the installation of a CFRP structural lining system, the following installation steps must take place: dewatering of the pipeline, set-up and maintenance of ventilation and environmental controls, initial cleaning of the pipe substrate, surface preparation, final cleaning of the pipe substrate, and verification of surface preparation. The by-pass system was put in-place, redundant safety controls were installed to allow for safe worker entry into the pipe, and an initial pressure wash of the pipe with a bleach solution was performed to sanitize the walls of the pipe and allow for surface preparation to take place.

The project requirements for surface preparation were as follows: a minimum surface profile of ICRI CSP 3 for the inner core substrate and near white metal blast (SP10) for the exposed steel pipe cylinder at the end terminations of the CFRP lining. The typical surface preparation methods used for preparation of the inner core substrate are sponge blasting, sand blasting and hydro blasting. Because either sponge blasting or sand blasting are necessary for the preparation of the steel substrates in the joint regions of the pipe, one of these techniques are more commonly used for surface preparation on smaller projects so that the same preparation method can be used for the inner core substrate and the joint regions. However, hydro-blasting using approximately 30,000psi water pressure becomes a more efficient inner core substrate surface preparation method for longer production runs and was utilized for this project because it was able to be run continuously through the length of the pipeline repair, without having to stop and clean a blast pot. Post cleanup for hydro-blasting was also simpler; the water ran downstream and was pumped out of the pipe.

Figure 5. Deterioration observed at the invert of the pipe

After surface preparation was completed, the entire surface of pipeline was inspected by the City's consulting engineer, the CFRP installer, and the CFRP system manufacturer to document any anomalies and address any unanticipated conditions. One region of the pipe, shown in Figure 5, had experienced such severe erosion damage at the invert that the entire inner core, steel cylinder and most of the outer core was missing for a region approximately 16 feet along the length of the pipe. While the CFRP lining system was designed as a stand-alone structural system to take internal pressure and all external loads without reliance on the host pipe for structural integrity, this severe level of distress required special detailing to make sure that water intrusion was stopped during lining installation to allow the CFRP materials to cure properly.

As part of the QA/QC program for the project the surface preparation and adhesion of the CFRP to the inner core substrate was verified using ASTM D4541 bond testing. This is a critical step because the installation of the CFRP liner system for pipeline rehabilitation is considered a bond critical application. Since the failure mode for the adhesion test is often a tensile failure within the inner core substrate, this test confirms both satisfactory adhesion of the CFRP to the inner core as well as the approximate tensile strength of the existing inner core concrete. CFRP panels were installed on prepared concrete substrate at approximately 60 LF intervals along the entire repair section. The adhesion tests were performed by the Engineer using 20mm test dollies installed on the adhesion test mock up areas, as shown in Figure **6**, prior to installation of the first layer of the CFRP lining system as the project progressed down the length of the pipeline.

Figure 6. Pull tests performed to verify adhesion.

INSTALLATION OF THE CFRP LINING SYSTEM

After completion of surface preparation, fabric layers are mechanically saturated with a two (2) part, 100% solids epoxy as shown in Figure 7. The mechanical saturator was calibrated several times per shift to ensure consistency in the epoxy to fabric weight ratio. The application of the CFRP lining system then involved applying a layer of thickened epoxy onto the prepared pipe substrate and applying epoxy saturated sheets of glass fiber and carbon fiber composites to the inside of the pipeline in the orientation designated per the project drawings. Once cured, the CFRP lining system is designed to take all of the internal pressure and external loads acting on the original pipeline without reliance on the host pipe. The design relies on the carbon fiber fabrics for structural integrity and the glass fiber fabrics to serve as an electrical isolation layer in any area where steel surfaces are exposed.

Figure 7. Mechanical saturation of the carbon fiber fabric.

Quality checks were performed after each layer of fiber was installed to confirm that proper development length was achieved, verify fabric alignment and spacing of layers was in accordance with project requirements, and check for any air bubbles. Any items requiring remediation were documented and addressed prior to continuing with the subsequent layer of CFRP.

CFRP laminates properties are dependent on the bond between each lamina. The type of bond required is based on the time lapse between which the layers are installed. If a new layer is placed upon a previously installed layer within the epoxy's cure cycle, a chemical bond is established; otherwise, a mechanical bond is required. To establish a chemical bond with the epoxy used on this project, new layers had to be installed within 72 hours of application. Due to the size of the repair, it was necessary to divide the CFRP installation into three (3) distinct construction regions, where each region was treated from an operational standpoint as a separate CFRP installation. The use of separate installation regions, along with careful planning of work sequences, allowed for subsequent layers of CFRP to be installed within the necessary 72 hour window to develop a chemical bond between the layers of CFRP. Figure 8 shows the typical application process for the CFRP layers.

Figure 8. Application of hoop direction layer of CFRP

Because the SWDS pipeline transports raw sewage, a chemical-resistant top coat specifically designed for sewage applications was installed over the CFRP structural lining system. Quality inspections were also performed by the Engineer on the top coat layer as shown in Figure 9 to check for any defects that required the application of additional top coats to insure conformity with the project requirements.

Figure 9. Inspection of the CFRP lining top coat

CONCLUSION

While the CFRP lining process has been used for structural upgrade of large diameter pipelines for over 15 years, this project is unique for multiple reasons. At 2,140 LF, the project's CFRP installation is the longest continuous CFRP installation performed to date in the United States. In addition, the complex by-pass system and other coordination efforts required to rehabilitate the sewer pipeline located in a heavily populated area presented additional unique project challenges. Due to the size of the project and additional project complexities, close coordination was necessary among the City, the engineer, and contractors for the CIPP and CFRP systems. The high level of communication was necessary to determine site logistics, access requirements, installation sequences and inspection of the CFRP work. This project demonstrated that CFRP is advantageous for structural upgrade of extended runs of large diameter sanitary sewer pipelines.

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Miami-Dade Implements Hybrid FRP Trenchless Repair System

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Abstract

In 2010, Miami-Dade Water & Sewer Department (MDWASD) implemented a comprehensive asset management program to increase the reliability of their large diameter pipeline system. The program includes inspection, prioritization and targeted replacement or rehabilitation of pipeline segments as needed. MDWASD utilizes several methods to address damaged or distressed pipelines including targeted fiber-reinforced polymer (FRP) structural repairs. Several critical Miami-Dade pipelines have had FRP systems installed at numerous segments as fully structural repairs. FRP repairs are trenchless and involve the use of carbon fiber and glass fiber-reinforcing fabric saturated in an epoxy matrix, then installed on the interior of the pipeline. Once cured, the FRP provides a standalone structural upgrade of the pipe, extending the lifecycle equivalent to replacement. Traditional FRP systems are typically used for segmental repairs when single or multiple non-contiguous pipe sections require rehabilitation. To address extended runs of pipe in a more cost-effective manner, a hybrid system has been designed utilizing continuously wound high strength steel reinforcement embedded in epoxy along with layers of FRP to form a structural upgrade system inside the existing pipe. In the summer of 2014, following an inspection, 13 pipe segments in need of structural upgrade were identified across a 3.5 mile stretch of Red Road. The 54-inch prestressed concrete cylinder pipe (PCCP) segments were selected as the most at-risk through MDWASD's risk ranking and prioritization process. The repair method elected for the 13 segments was installation of FRP as a stand-alone structural upgrade. Also, as part of the project, Miami-Dade set aside 3 of the 13 segments for installation of the Hybrid FRP System, also implemented as a stand-alone structural upgrade. The Hybrid FRP System installation process includes surface preparation, application of FRP and installation of the steel tensile reinforcement. The steel is installed using specialized equipment which places the reinforcement in the hoop direction onto the pipe's interior surface. The general design approach utilizes the steel reinforcement

for resisting hoop direction design requirements and the FRP component of the system for resisting longitudinal design requirements. The steel is typically placed between layers of FRP with the steel reinforcement set into an epoxy putty for system continuity and protection. The Miami-Dade Hybrid FRP installation was implemented successfully within the allotted schedule. Readers will learn detailed information about MDWASD's experience selecting and implementing the Hybrid FRP repair at three (3) segments of 54-inch PCCP. The benefits and limitations of the system will be explored including design, materials selection, installation and the appropriate quality control measures.

BACKGROUND

Miami-Dade Water & Sewer Department (MDWASD) operates one of the largest public utilities in the United States. MDWASD services approximately 2.3 million people, the state's highest population. The customer base consists of outlying areas of Miami-Dade County, 485,000 service connections and 15 municipal wholesale customers.

MDWASD's service area consists of over 7,900 miles of water mains ranging in size from 2 inches to 120 inches in diameter spread across 400 square miles. MDWASD uses predominantly Prestressed Concrete Cylinder Pipe (PCCP) for large diameter transmission mains and has over 100 miles of PCCP that is 48-inches and larger. Much of the PCCP is located under major roadways in densely populated areas.

After a series of high profile catastrophic failures in 2010 and 2011, similar to the one shown in Figure 1, MDWASD began to focus on pipeline management and developed a comprehensive asset management program. The objective was to implement a program to minimize disruptions to residents and other customers. MDWASD had to develop a program that balanced upgrading and replacing pipelines to the region's increasing base of customers.

Figure 1 – Ruptured 54-inch PCCP water transmission main

MIAMI-DADE'S INRASTRUCTURE ASSESSMENT AND REHABILITATION PROGRAM

To address these needs, MDWASD established the Infrastructure Assessment and Rehabilitation Program (IAARP). The program consists of routine inspections of the pipeline inventory on a rotating basis. As part of IAARP, MDWASD adopted industry best practices which include precision inspection, replacement and structural upgrade for its large diameter PCCP inventory.

Through specialized electromagnetic inspection services provided by Pure Technologies, MDWASD is able to evaluate their entire large diameter PCCP system to identify distressed and high-risk pipe segments. The inspection technique utilized is able to identify broken prestressing wires throughout each segment. When a large enough number of prestressing wires break on a given segment of PCCP structural integrity of the segment is compromised. Utilizing inspection methods that pinpoint which pipe segments have broken wires allows MDWASD pipeline systems to be repaired or replaced before failures cause unscheduled and costly shutdowns.

After an inspection of a pipeline run is completed, MWASD works with their consulting engineers to analyze the data and develop a failure risk analysis. This process includes taking into account all of the factors for each specific pipe including prestressing wire pitch and spacing, cylinder thickness, concrete core thickness, along with internal and external loads acting on the pipe. From this analysis the pipes inspected and identified as distressed can be ranked into groups for near term replacement or repair, mid-term and long term which require monitoring. Following this, the decision making process for how to address near term repairs is completed.

Many of MDWASD's pipelines are located underneath major roadways which make trenchless rehabilitation techniques advantageous over dig and replace or other repair methods requiring excavation. In cases where rehabilitation is the correct technical solution, MDWASD addresses the identified high risk pipeline segments with targeted structural repairs installed without excavation.

For the past 5 years MDWASD has utilized high-strength fiber-reinforced polymer (FRP) as a method to provide structural upgrade to specific pipe segments identified during inspections and failure risk analysis. Many projects have been completed using FRP for PCCP lines ranging from 48 to 96-inches, and a typical installation is shown below in Figure 2. MDWASD has established an emergency response team (ERT) of prequalified specialty contractors who can perform this work through an on-call process.

Figure 2 – Typical FRP installation

Following an inspection and failure risk analysis in mid-2014, MDWASD moved forward with repair of 13 segments of 54-inch PCCP. Ten (10) of the segments were repaired using FRP and MDWASD elected to implement the StrongPIPE hybrid FRP system for three (3) of the 54-inch segments.

HYBRID FRP SYSTEM OVERVIEW

Figure 3, below, demonstrates the composition of the Hybrid FRP system used by MDWASD on their recent project. The system consists of high strength steel reinforcement continuously wound around the inside circumference of the pipe. The steel reinforcement is embedded in an epoxy matrix and sandwiched between layers of glass or carbon FRP reinforcement that provide the required strength in the longitudinal direction.

The “sandwiched” composite structure is engineered to meet all design requirements acting on the system in both the circumferential and longitudinal directions. The continuous circumferentially installed steel reinforcement is intended to provide the main reinforcement in the hoop direction, while the FRP materials are oriented primarily in the longitudinal direction to resist the longitudinal loads acting on the system.

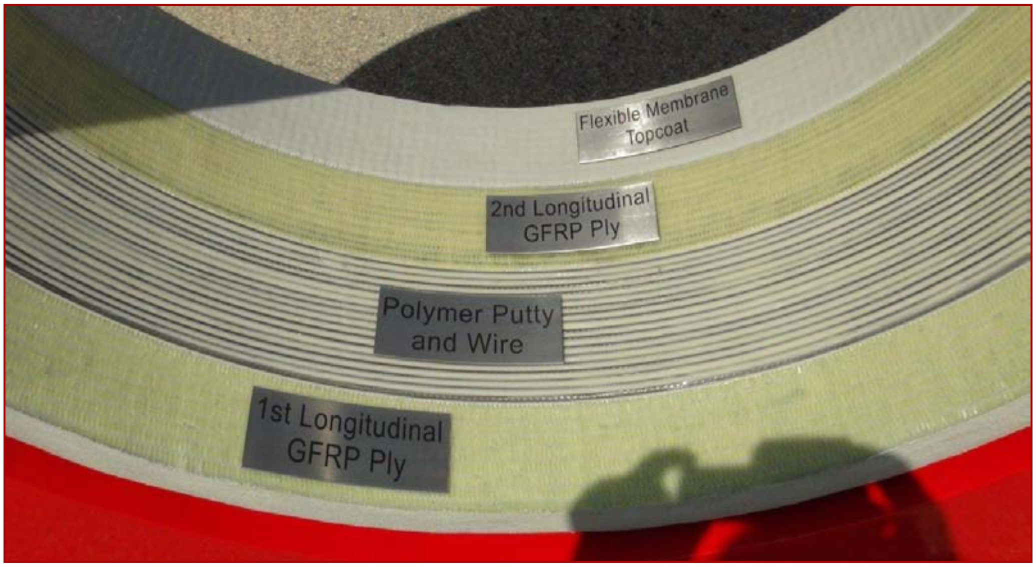

Figure 3 – Basic composition for Hybrid FRP system

The purpose of the top and bottom longitudinal layers of glass and/or carbon FRP reinforcement is twofold: (1) to provide resistance of the Hybrid FRP System against longitudinal stresses resulting from internal pressure thrust, Poisson's effect from internal pressure, and temperature variations; and (2) to provide additional environmental protection for the high strength steel so that the steel is fully protected from contact with water. Glass and carbon FRP reinforcement have been widely used in the strengthening of concrete structures since the early 1980's. Its use extended to PCCP rehabilitation in the United States in the mid-1990s, and it has been steadily increasing in the last 15 years.

The Hybrid FRP System has been designed to provide an economical solution for short as well as long runs of repair, which have been typically considered to be cost prohibitive for FRP applications. The system is very appealing for the rehabilitation of PCCP due to its lower cost compared to other available methods, and because it involves the use of high strength steel, which is a widely accepted material for structural upgrade applications.

HYBRID FRP SYSTEM DESIGN FOR MDWASD 54-INCH PCCP

Design of both the FRP System and the Hybrid FRP System were tailored to operating conditions and took into account several factors including internal pressure, transient pressure, vacuum pressure and all external loadings. Miami-Dade typically has all structural upgrades on their large diameter PCCP designed for 150psi operating pressure with operating plus transient pressures up to 225psi. In addition, designs incorporate full vacuum as well as soil cover along with ground water up to the top of the soil cover. For vehicular loads, MDWASD accounts for HS-20 vehicular loading for all pipe segments, since most of their pipelines run under roadways throughout the city.

The Hybrid FRP System for pipe nos. 456, 457 and 459, 20LF segments of PCCP was designed as fully structural, with no reliance on the host pipe. It consisted of multiple layers of Carbon FRP (0.08-inch thickness), Glass FRP (0.04-inch) and 260 ksi steel reinforcement (0.208 inch diameter, 30 steel reinforcements per foot) oriented to meet design requirements.

INSTALLATION OF THE HYBRID FRP SYSTEM FOR MDWASD 54-INCH PCCP

The Hybrid FRP System installation for MDWASD 54-inch PCCP took place using trenchless methods which included manned entry into the pipeline structure. Prior to entry, proper confined space procedures were implemented to meet OSHA requirements regarding work taking place within a confined space and appropriate ventilation was installed. Once jobsite safety was addressed, the system installation commenced.

The project set-up for the Hybrid FRP System required a minimal topside footprint which included a truck mounted unit for the steel reinforcement installation. The truck unit is shown in Figure 4 below.

Figure 4 – Truck mounted unit for Hybrid FRP System steel reinforcement installation

The first step of pipeline operations completed for the Hybrid FRP System installation at the 54-inch PCCP for MDWASD was surface preparation. The inner core concrete was abrasively blasted to roughen the surface to a minimum concrete surface profile (CSP) of 3 as defined by ICRI 310.2 guideline. At the termination of the Hybrid FRP system, the joints of the PCCP were chipped out to expose the bell and spigot steel and the steel substrate in joint areas was prepared to near-white metal condition as defined by SSPC-SP10/NACE No.2. Figure 5 below shows the prepared concrete substrate.

Figure 5 – Prepared concrete substrate – MDWASD 54-inch PCCP

Following surface preparation the substrate was tested for proper bond using the ASTM D4541 procedure and upon confirmation installation of the longitudinal layer of Carbon FRP was installed. The FRP materials were prepared topside using a mechanical saturation machine and installed over each repair segment in the longitudinal direction. After the Carbon FRP layer was installed, per the design requirements a layer of Glass FRP was installed.

The next step in the Hybrid FRP System was the installation of the spirally wound steel reinforcement. The truck mounted unit, shown above in Figure 4, fed the steel reinforcement in a spiral fashion into the pipe where it was placed using automated equipment specially developed for the in-pipe installation. The steel reinforcement installation equipment, shown in Figure 6 below, was dismantled and reassembled in the pipe, allowing for the equipment to fit through the 16-inch by 18-inch manholes typically available for entry into MDWASD's pipelines.

Figure 6 – Hybrid FRP System – Steel reinforcement installation

Following placement, the steel reinforcement was then covered with a specialized thickened epoxy with a putty-like consistency. As per the approved design, a layer of Glass FRP was applied following the steel reinforcement.

Termination details were used at the ends of the system to prevent water migrating through any cracks within the inner core concrete at the termination of the repair. To achieve this, the mortar in the joint as well as a small portion of the inner concrete core at the end of the pipe was carefully removed to expose the steel cylinder and to create a transition region using a wedge built up with epoxy mortar. The system was transitioned at the termination and bonded directly onto the bell or spigot steel and steel cylinder. Stainless steel expansion rings were then installed in the joint terminations to guarantee full intimate contact. Once the rings were installed, the joint region was filled in flush with epoxy mortar.

QUALITY CONTROL PROGRAM FOR THE HYBRID FRP SYSTEM

Quality Control during installation of the Hybrid FRP System was a critical component to project implementation and required close coordination with MDWASD and their consulting engineer. The process included QA/QC checks typical of standard FRP installations plus a group of additional steps for the steel component of the Hybrid system.

QA/QC for FRP component. During the installation and cure time of FRP, environmental conditions are monitored and tracked. These include temperature and humidity. As materials are prepared for installation there are steps for verification which include confirmation of the mixing procedure for epoxy and documentation of calibration of rollers within the mechanical saturation equipment, shown in Figure 8.

Figure 8 – Mechanical saturation equipment

The QA/QC process at the point of installation within the pipe includes several steps starting at surface preparation. As mentioned in previous sections the substrate is checked for bond using the ASTM D4541 process.

As installation of the Hybrid FRP System takes place, the materials have QA/QC checks which include verifying proper alignment of the FRP material and confirming that appropriate overlaps within the FRP layers are achieved per the approved project drawings. Following installation of the fiber layers, inspections are performed to check for air bubbles and insure that intimate contact is achieved between the pipe substrate and the FRP layers.

The FRP is also tested for tensile properties using the ASTM D3039 testing protocol. Sample panels are created using the same material installed in the pipelines. The samples are sent to a laboratory and tested to confirm the properties meet or exceed design values utilized.

QA/QC for Steel Reinforcement component. The second major component of the Hybrid FRP System is steel reinforcement and there are several QA/QC steps associated with this portion of the installation. As the steel is delivered to the jobsite, the lot numbers of the steel are verified. A visual check of the steel takes place as it is placed onto the pipe surface. In addition, the spacing of the steel reinforcement – confirmation that the correct numbers of steel reinforcement wires are placed within each lineal foot of pipe takes place. There is also a check to determine that the minimum and maximum spacing between each steel wire in relation to adjacent steel meets project requirements. This is shown in Figure 9.

Figure 9 – Verification of steel wire placement

After the steel is installed, the thickness of epoxy used to encapsulate the steel is verified to insure proper coverage and protection of the steel wire.

CONCLUSION

The Hybrid FRP System installation was completed successfully and MDWASD is exploring the use of this system for upcoming projects. MDWASD produces approximately 350 million gallons of water every day through its pipeline system for its customers. Through the development and implementation IAARP, MDWASD effectively manages its inventory of pipelines. As a result, MDWASD ensures pipeline reliability, manages limited resources, and improves the life of residents.

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Composite versus Stand-Alone Design Methodologies for Carbon Fiber Lining Systems

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Abstract

Fiber Reinforced Polymer (FRP) composite lining systems are used by major municipalities throughout the United States to structurally rehabilitate and upgrade large diameter pipelines. For internal Carbon Fiber Reinforced Polymer (CFRP) lining systems addressing prestressed concrete cylinder pipes (PCCP), there are two design approaches utilized relative to interaction with the host pipe structure. These approaches are referred to as stand-alone and composite. For a stand-alone design, the carbon fiber takes 100% of the loads acting on the pipeline system with no reliance on the host pipe for structural integrity. Composite designs rely on the carbon fiber lining system and inner concrete core of the PCCP to interactively provide a structural system to resist the loads. A composite design approach relies on the inner core to resist bending and buckling due to external loads such as soil cover, water table, vehicular loads and vacuum pressure. When applicable, this type of design can be more cost-effective because the amount of carbon fiber materials utilized can be less than stand-alone design. This paper presents design limit states and includes information from recent research, development, and testing. It discusses factors to be considered, potential challenges and best practices for determining stand-alone versus composite designs for carbon fiber lining systems.

BACKGROUND

Over the past more than 15 years, Fiber Reinforced Polymer (FRP) composite materials have been utilized with increasing frequency for internal structural rehabilitation and upgrade of pipelines. The overall process involves surface preparation of the internal pipe substrate followed by manual application of layers of unidirectional carbon fiber fabrics (Figure 1) which have been saturated with a two part epoxy directly prior to installation using a calibrated mechanical saturator.

Figure 1. Typical process for Installation of CFRP inside a pipeline

The layers of carbon fiber fabric are oriented in the longitudinal and the circumferential directions and are designed to resist the structural demands acting on the pipeline. Depending on the design approach, the CFRP liner can be designed as a stand-alone system or a composite system which relies on the host pipe for partial structural strength.

CFRP liners are commonly used to structurally rehabilitate prestressed concrete cylinder pipeline (PCCP) segments which have been identified as distressed. An embedded-cylinder type (ECP-type) PCCP, the type of PCCP used for larger diameters pipelines, is composed of an inner concrete core, a steel cylinder, an outer core, prestressing wires over the outer core, and a protective mortar coating (Figure 2). A common failure mode of PCCP is breakage of the prestressing wires within individual PCCP sections.

Once enough prestressing wires break, the concrete core in the region near the broken wires is no longer in compression and can crack, exposing the steel cylinder to ground water and thus, causing corrosion. The condition of the host pipe is critical in determining what extent of the host pipe, if any can be taken into account in the CFRP lining design. Since the steel cylinder, outer core, and prestressing wires are debonded from the inner core, only the inner core can be relied on in composite CFRP design for addressing distressed PCCP segments.

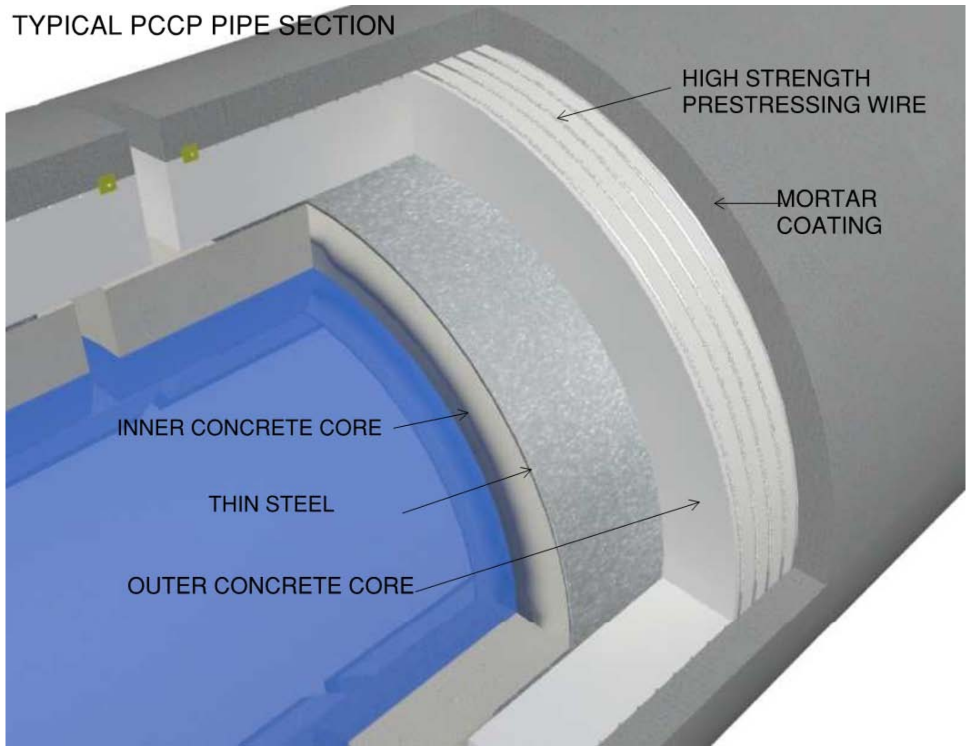

Figure 2. Components of an ECP-type Prestressed Concrete Cylinder Pipe Section

DISTRESS LEVEL OF THE HOST PIPE

As part of the CFRP lining design process, the overall distress level within the host pipe is considered. These levels of degradation are defined in a draft AWWA standard for CFRP rehabilitation and strengthening of PCCP as Non-Degraded Pipe, Degraded Pipe, and Severely Degraded Pipe.

a) A non-degraded host pipe is taken into account in the design when there is no known damage to the PCCP segment and the CFRP liner is added due to load increases acting on the pipeline (live load, earth load, pressure, etc.). Based on the good condition of the pipe, the CFRP system can be designed as composite action with the entire pipe wall thickness.

b) A host pipe is defined as a degraded pipe when the PCCP has some broken wires and the outer concrete core may be also cracked and softened, but any minor cracking of the inner core can be repaired and the inner core is still circular. The host pipe is expected to continue to degrade with time after the CFRP repair is in place. Since additional wire breakage, outer core cracking, and corrosion of steel cylinder are anticipated over time, the CFRP repair of degraded pipe can be based on either composite action of the host pipe inner core reinforced with CFRP laminate or stand-alone CFRP liner.

c) A severely degraded host pipe consists of PCCP with a non-circular inner concrete core showing multiple wide cracks as well as an uneven internal surface with ovality or waviness. Pipes with this level of severe distress require special design consideration and additional attention should be given to determining applicability of CFRP lining for these applications.

DESIGN PROCESS FOR FRP REHABILITATION OF PCCP SEGMENT

Design process used for FRP Rehabilitation of PCCP at WSSC consists of several steps depicted in below the diagram (Figure 3).

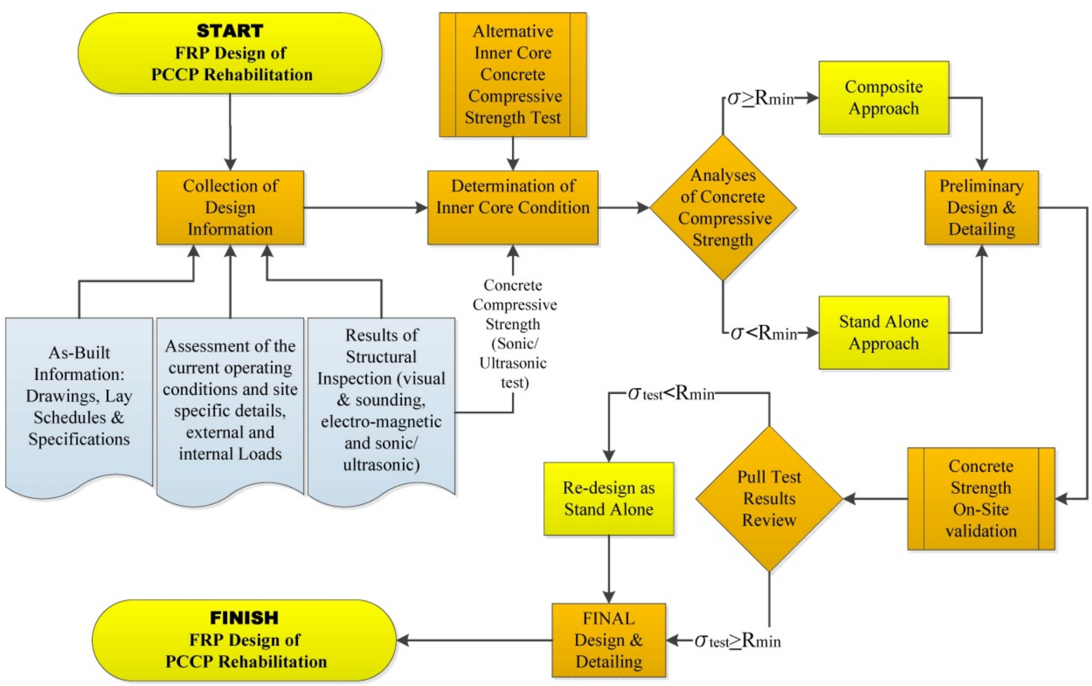

Figure 3. FRP Design Process

The design process involves collection of the design information, including as-built drawings, lay schedules and pipe specifications, results of structural pipe inspection (visual and sounding, electro-magnetic, sonic/ultrasonic, etc.) and assessment. Since stand-alone design for the large diameter PCCP most likely will utilize more layers of the FRP material than the composite design method, the design process may start with validation of the less expensive composite method which relies on the existing condition of the substrate, i.e. inner core concrete. Determination of the concrete condition is one of the most critical components needed for the "composite versus stand-alone" decision since inner core concrete compressive strength is used for the estimate of the FRP-to-substrate bond. Should adhesion bond, σ, be less than the minimum allowed bond value (R_{min}) per AWWA [5] such that $\sigma < R_{min}$, a stand-alone design approach may be used. In order to confirm the adhesion bond and therefore determine applicability of a composite design approach, on site pull-off testing must be performed.

Determine Condition of Inner Core

In order to determine whether the inner core is capable of being used in a composite CFRP design, a condition assessment is performed to evaluate the level of deterioration that has taken place. Several methods are used at WSSC for determining condition of the inner concrete core include visual and sounding, adhesion testing, sonic/ultrasonic, and rebound hammer testing.

Visual and sounding inspection of a pipe involves a trained inspector looking for signs of distress within the pipe which include cracks within the inner core, damaged joints, areas with severe pipe ovality, and concrete spalling. One sign of severe distress in a PCCP section involves longitudinal cracks within the inner core, which could indicate loss of prestress due to broken wires.

Tests to estimate the compressive strength of concrete include sonic/ultrasonic inspection which can be performed as a part of the structural assessment [8], and the rebound hammer test (i.e. Schmidt hammer test) per ASTM C805 [4] which involves a spring loaded hammer hitting a steel plunger, which is in contact with the concrete as shown in Figure 4. Once the concrete is impacted by the defined energy, the hammer's rebound distance is measured. This rebound hammer can be used to determine the concrete's compressive strength using the manufacturer's conversion chart [9].

Figure 4. Evaluation of Inner Concrete Core via Rebound Hammer Test

In order to validate design based of the estimated inner core concrete values, adhesion tests must be performed in accordance with ASTM D4541 [3]. Adhesion tests are a part of the typical QA/QC process for the CFRP lining process. A common failure mode observed in the adhesion tests is tensile failure within the inner concrete core substrate so the results from adhesion testing provide a measure of the tensile strength of the concrete core (Figure 5). Since the tensile strength of concrete is approximately 10% of concrete's compressive strength, the compressive strength of the concrete can be approximated through use of adhesion tests on the inner core substrate. The calculated compressive strength for the inner core concrete can be checked against the minimum required values used in the design ($\boldsymbol{\sigma} < R_{min}$).

Figure 5. Pull Test per ASTM D4541

DESIGN APPROACH

CFRP systems are designed using a Load and Resistance Factor Design (LFRD) approach (AWWA draft standard), where factors are applied to applied loads and material properties to account for uncertainties within the design assumptions.

As part of this design approach, design limit states are analyzed separately and the CFRP lining design is governed by the limit state that has the lowest demand to capacity ratio for the particular design scenario. Various limit states are accounted for in the design depending on whether a composite or stand-alone system is being considered.

Stand-Alone Design

For stand-alone design, the following limit states must be considered:

- Rupture of CFRP laminate in the circumferential direction due to internal pressure.

- Rupture of CFRP laminate in the circumferential direction due to bending of empty pipe.

- Rupture of CFRP laminate in the circumferential direction due to combined pressure and bending due to gravity loads.

- Buckling of CFRP laminate in the circumferential direction due to external loads and pressures and internal negative pressure

- Rupture of CFRP laminate in the longitudinal direction due to pressure induced thrust, Poisson's effect of internal pressure, and temperature changes in the pipe.

- Shear bond failure of the CFRP at pipe ends.

- Rupture of CFRP laminate in the longitudinal direction due to radial expansion of pipe in broken wire zones.

- Compressive failure of CFRP laminate in the longitudinal direction due to radial expansion of pipe in broken wire zones.

- Buckling of CFRP liner in the longitudinal direction due to temperature increase.

Composite Design

Composite design can be applied in situations where the host pipe is classified as non-degraded or degraded.

When a PCCP section is considered degraded and only the inner concrete core is taken into account in the CFRP design, the following additional limit states are addressed:

- Debonding of CFRP from the concrete inner core under one of the following circumstances:

  - Shear between the CFRP and the concrete inner core.

  - Excessive radial tension.

  - Concrete core crushing from gravity loads, in absence of internal pressure.

In situations where the host pipe is considered non-degraded and the CFRP lining is utilized to upgrade or strengthen the existing pipe, the entire wall thickness may be considered in composite action.

In the design process, it is initially assumed that the CFRP lining is acting compositely with the concrete inner core. Since stand-alone designs typically require higher layer counts than composite designs, in order to not unreasonably increase an amount of CFRP layers and consequently the cost of the repair, the design may start as composite. The bond between the CFRP liner and the inner core is checked and if any of the limit states are not satisfied, then the system must be designed as a stand-alone.

*Recent Testing Affecting CFRP Lining Design*s

Over the past several years, significant research and development efforts have taken place impacting best practices regarding designs of CFRP linings. One of the major testing programs was completed in conjunction with the Water Research Foundation (Zarghamee et al.) [10]. The testing included full scale external load tests and internal pressure tests.

External load testing, such as that recently completed (as shown in Figure 6), assists in better understanding of the CFRP and inner core composite action mechanism and ultimately helped validating the design approach which relies on the inner core for composite CFRP design.

Figure 6. Water Research Foundation Testing Setup (Zarghamee et al, 2013)

One of the most significant findings in recent testing is that watertightness of the CFRP lining is critical to long term performance, whether in a stand-alone or composite design approach. The termination details must be effective in preventing pressure build-up behind the CFRP liner. It was determined that preparation of the steel substrate at the pipe ends (for PCCP) is to be completed in a manner which ensures that material bonding is not compromised.

Along with its importance at the terminations, watertightness of the entire CFRP liner is a recent point of focus with regard to permeability. Best practice for CFRP liner materials now includes validation of watertightness for different laminate designs through testing and inclusion of watertightness provisions within each CFRP design.

CONCLUSION

The composite design process is considered a typical design concept for CFRP lining of PCCP. In order to establish feasibility of a composite design, the pipe must be verified through inspection to determine the condition of the inner core substrate within the host pipe. When composite designs are feasible, they have the potential to help reduce the overall layer count for the CFRP lining system, thereby helping pipeline owners further extend rehabilitation dollars. When composite designs are not feasible, the CFRP lining system can be designed as a stand-alone system to take all loads without reliance on the host pipe for structural integrity.

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Better Data Equals Better Decisions: New Developments in Multi-Sensor Condition Assessment Technologies

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Abstract

This paper describes the development and successful applications of a CCTV, LIDAR and sonar based pipe inspection system that is robust to gather quantitative data for critical underground pipe condition assessment. The system that can be deployed on a ROV or on a float and produces accurate cross-sectional analysis and sediment volume. This capacity is increasingly critical in large diameter pipes with high level of flow. The system employs a time of flight LIDAR that is accurate to 1/16th of an inch. Results from recent projects are discussed in detail. The Huntington Trunk sewer in Abbotsford, British Columbia, Canada is a critical line in the municipality's waste-water system. It is a PVC and HDPE pipe that also carries sewage from Sumas, WA. Pipe diameters vary between 10" and 27" with highly variable flow conditions. Hard to access, off street manholes located in a swamp and on a railway right of way created challenges during deployment. The robust, yet modular SewerVUE multi-sensor pipe inspection system (MPIS) was repeatedly reconfigured during the project to accommodate the challenging site conditions. The sonar results provided accurate sediment volumes and cross sectional restrictions. The Quai George Gorse combined sewer in Boulogne-Billancourt, a southerly suburb of Paris, France is a critical interceptor in the SEVESC operated collection system. This 2200 mm wide and 2700 mm high, irregular shaped ("cunette avec banquettes") reinforced concrete pipe runs parallel to the Seine river and experiences wet weather overflows during extreme rainfall events. The primary objective of the survey was to quantitatively measure sediment volume and distribution within a 1275.8 m long section. This paper presents the methodology and the results of the inspection. Advanced pipe condition assessment technologies, such as the CCTV, LIDAR and sonar system described in this paper are cost-effective, non-destructive methods that are able to help better refine estimated remaining life of an interceptor, accurately determine overall severity of pipe degradation, as well as provide a basis for improved cost allocation and timing of rehabilitation efforts.

INTRODUCTION

Obtaining quantitative data which allows for objective assessment of pipes is of increasing interest to engineers, contractors, and municipalities. Conventional closed-circuit television inspection technologies cannot adequately meet this need due to the subjective and imprecise nature of the assessment process. Laser profiling is an emerging technology that has been shown to provide precise measurements of pipe parameters such as ovality, unobstructed cross-sectional area, pipe deformations, lateral size, offset joints, and flow levels.

Accurate pipe dimensional data is especially critical for CIPP (cured in place pipe) design engineers, as ovality is one of the main influence factors in the CIPP liner design equation as specified in ASTM F1216-03 (Dettmer et al., 2005). With accurate measurements, CIPP liners can be designed more cost effectively by reducing the required thickness of the liner.

Municipalities and contractors have also shown interest in the verification of the dimensions of newly installed CIPP liners or pipes (Shelton and Travis, 2012). Having an accurate pipe-wall profile of both pre- and post-installation would guarantee that the liner was designed correctly and help determine which party would be responsible in the event of a liner failure. For example, if the liner was designed for a known geometry but was installed incorrectly by a contractor, accurate pipe profile data could verify the fact that the contractor is at fault. Conversely, if the liner was installed correctly as specified by the design engineer, and the liner fails, then the fault would lie with CIPP liner designer, not the contractor (Dettmer et al., 2005).

The laser profiling concept as well as its inherent measurement errors are described by Dettmer (2007) and by Dettmer et al. (2005). There are several commercially available models on the market. Their reliance on accurate calibration and unreliable field accuracy was pointed out in a seminal paper by Shelton and Travis (2012).

The approach outlined in this paper employs LIDAR (LIght and raDAR) an optical remote sensing technology that measures properties of scattered light to find range and/or other information of a distant target. The prevalent method to determine distance to an object or surface is to use laser pulses. Like the similar radar technology, which uses radio waves, the range to an object is determined by measuring the time delay between transmission of a pulse and detection of the reflected signal. The SewerVUE MPIS's LIDAR data is correlated with an onboard inertial navigation system (INS) that uses a computer, motion sensors (accelerometers), and rotation sensors (gyroscopes) to continuously calculate via dead reckoning the position, orientation, and velocity (direction and speed of movement) of the inspection platform without the need for external references. This technology is commonly used on vehicles such as submarines and guided missiles and is specially adapted for the use of multi-sensor inspections for underground infrastructure surveys where LIDAR is utilized and location and time measurement data is necessary. The multi-sensor system can be deployed from an autonomous robot (or ROV) or from a floating platform. Successful applications for each are described in the following case studies.

METHODOLOGY

LIDAR Theory

LIDAR (also written Lidar or LiDAR) is a remote sensing technology that measures distance by illuminating a target with a laser and analyzing the reflected light. Although erroneously considered to be an acronym of LIght Detection And Ranging, the term Lidar was actually created as a portmanteau of "light" and "radar".

Lidar uses ultraviolet, visible, or near infrared light to image objects. It can target a wide range of materials, including non-metallic objects, rocks, rain, chemical compounds, aerosols, clouds and even single molecules. A narrow laser-beam can map physical features with very high resolution. Wavelengths vary to suit the target: from about 10 micrometers to the UV (ca. 250 nm) range.

Understanding how each laser profiler works, its advantages and disadvantages, is imperative for engineers in charge of pipe specification, installation, maintenance or testing. Output from laser profiling systems can vary greatly. For example, the difference in results even from the same ring laser profilers operated by different contractors can be significant (Shelton and Travis, 2012). Municipalities and engineers must carefully assess the repeatability, accuracy and calibration of the employed systems.

Continuous-ring profilers use a planar laser whose light rays emanate radially outward in a continuous fashion from a fixed focal point. The laser plane is perpendicularly aligned to the pipe axis. Incident rays on the interior wall readily illuminate its orthogonal cross section. Using a calibrated high-definition digital camera, the illuminated ring is imaged along the pipe's axis and then analyzed. Because of the camera calibration, the digitized image contains usable spatial information (known relation between pixels and actual distance). By counting the number of pixels from the center of the pipe to the incident laser, many radial distance measurements are obtained simultaneously along the pipe wall. When the camera-laser is in motion, the camera frame rate assures that the illuminated ring is imaged at fixed intervals along the pipe (Salik and Conow, 2012).

LIDAR systems use a scanning laser that moves back and forth in a single plane. Distance measurements are acquired by measuring the time it takes for the laser to bounce off of a target and return to its origin. Because the light propagation speed is constant, distance can be determined from the "time of flight." The scanning motion results in a plane that projects along the interior pipe wall. Because the laser's angular step remains constant, the orthogonal measurements from the pipe's center to the wall are taken only two at a time (per sweep), but at many non-uniform distances from the robot. When placed in rotation, many pairs of distances are acquired so that a ring of measurements is formed. This measurement ring forms a 2-D cross section, and with many sections obtained simultaneously, a 3-D pipe profile can be created (Salik and Conow, 2012).

ABBOTSFORD, BC, CANADA

The Huntingdon Trunk Sewer is a 10 inch to 27 inch (250mm to 675mm) diameter PVC and HDPE and sewer pipe in the City of Abbotsford's (City) collection system. This is a critical line since it also carries sanitary sewage from Sumas, WA. Typical issues include FOGs and high sedimentation reducing capacity and causing SSOs. The objective of the inspection was to determine the condition of the inspected pipes by mapping out the accumulated sediments at the bottom of the pipe. The inspected sections were located between Farmer St and McConnell Rd (Figure 1). The total inspected length was 5,589 ft (1863 m).

The inspection took place between March 26 and April 1 2014. SewerVUE crew was assisted by the City and a local contractor for traffic control and site safety. This paper presents the methodology and results of the inspection.

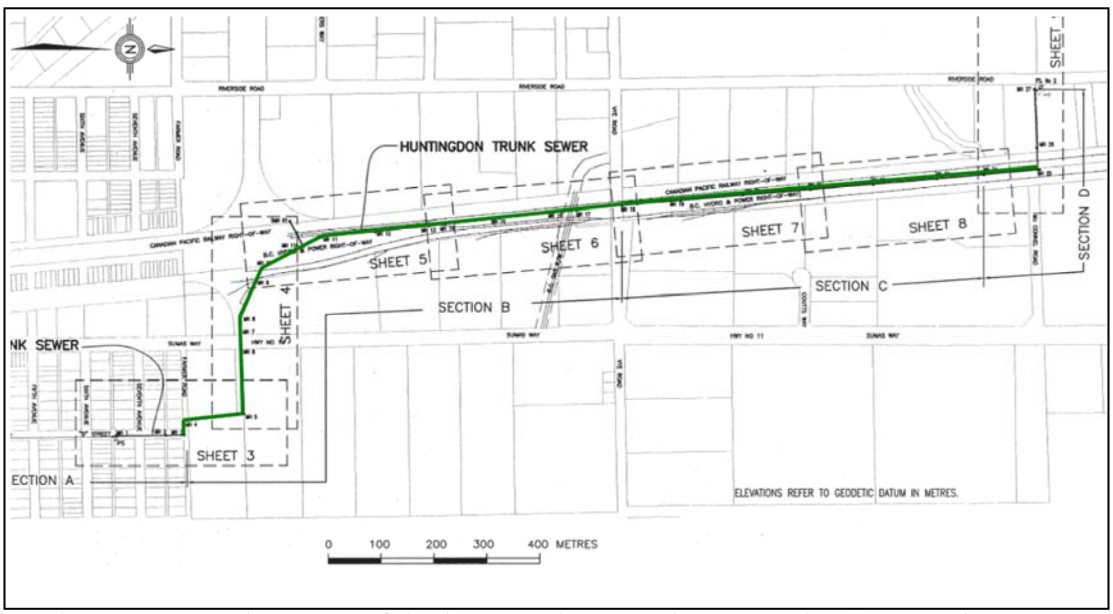

Figure 1. Overview map of the inspected sewer pipes, Huntingdon Trunk Sewer, Abbotsford, BC, Canada.

SURVEY EQUIPMENT

The SewerVUE Multi-sensor Pipe Inspector System (MPIS) is a float based inspection system that uses visual and quantitative technologies (CCTV, LIDAR, and Sonar) to inspect the condition of underground pipes. This tethered, modular and customizable second generation MPIS was attached to a light weight and mobile 500 feet (150 m) long tether cable and reel. This allowed truck independent deployment and operation which was critical for the project since most of the manholes had no vehicle access (Figure 2).

The floating platform was modified to fit through 18" (450 mm) pipe sections. CCTV, LIDAR and sonar data were acquired simultaneously in both in and out directions. The pipe diameter of the inspected sections varied between 18 and 27 inches (450 mm and 600 mm), pipe material was PVC, HDPE and steel. First a guide rope was installed then the inspection platform was winched through. A total of 5589 ft (1863 m) was inspected.

Figure 2**.** Deployment of and data collection with the second generation SewerVUE Multi-sensor Pipe Inspection System on a railway right-of-way in Abbotsford, BC, Canada.

The primary objective was to measure the height, volume and distribution of the sediment for subsequent cleaning and maintenance. By quantifying the sediment distribution over time the City can better maintain the pipe, locate the primary source of the sediment and take corrective actions.

In total, 2,562 ft^3 of debris was detected along the 5,589 ft of the Huntingdon Trunk Sewer. The average cross-sectional restriction for the sections of pipe ranged from 3.9% to 28.5% with an overall average of 12.6%. An example of the pipe cross-sections are shown in Figure 3. The cross-sectional restriction did not appear to correlate with pipe diameter or material.

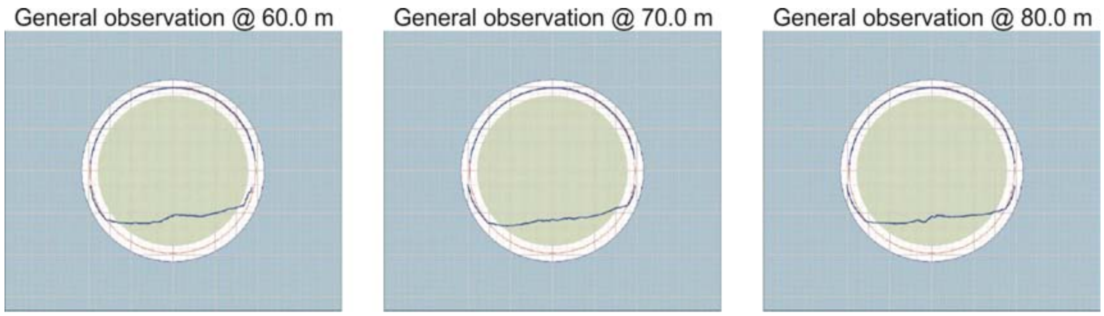

Figure 3. Showing an example of pipe cross-sections from the Huntingdon Trunk Sewer. The line represents sediment level in the pipe. This section had an average cross-sectional restriction of 9.0%.

BOULOGNE-BILLANCOURT, FRANCE

The 2200 mm wide and 2700 mm high irregular shaped reinforced concrete interceptor runs parallel to the Seine River in the municipality of Boulogne-Billancourt in the outskirt of Paris, France. This combined sewer is a critical line in the sewer network of the municipality that experiences wet weather overflows directly to the environmentally sensitive Seine River during extreme rainfall events. Therefore, monitoring the sediment level and volume is critical for the efficient operation of the sewer. Previously used methods such as measuring sediment depth with sticks via manned entry provided only point data and are both inaccurate and potentially dangerous to operators.

SEVESC, the organization in charge of the maintenance of the pipe was looking for safer and more efficient ways to monitor the condition of the pipe. They contracted SewerVUE to deploy its MPIS. The primary objective of the survey was to quantitatively measure sediment volume and distribution within a 1275.8 m long section. Manholes spaced at regular intervals provided relatively easy access, while offset manholes provided some operational challenges. The inspection was completed in late November 2014. The inspection took place while the pipe was in service flowing between 50 and 70% of capacity.

The SewerVUE's long range multi-sensor pipe inspection (MPIS) technology combines state of the art data collection and analysis with proprietary processing and reporting software. The float based inspection platform is outfitted with high definition CCTV, LIDAR and sonar sensors and has a 4000 ft maximum deployment capability. LIDAR measurements determine the exact size and shape of the pipe and provide quantitative assessment of deformation and corrosion. Sonar accurately profiles the pipe below the flow line and calculates the sediment and debris volume in the pipe. The system is customizable and can be deployed through a 18 inch manhole and can inspect any pipe size over 18 inches. Bypass pumping is not required. Inspection reports provide integrated and quantitative corrosion and debris measurements, 180 degree virtual pan/tilt/zoom function, video, laser and sonar flats.

Figure 4. The SewerVUE MPIS and field crew before deployment in Boulogne-Billancourt.

A total of 285.7 cubic m of sediment was found in the inspected 1214.7 m long pipe section. In some sections the sediment was up to the top of the central trough ("cunette"). Lidar profiling did not detect any significant corrosion. The reported sediment volume and distribution helped mangers of the pipe to prioritize targeted cleaning and reinforced the need for regular condition monitoring.

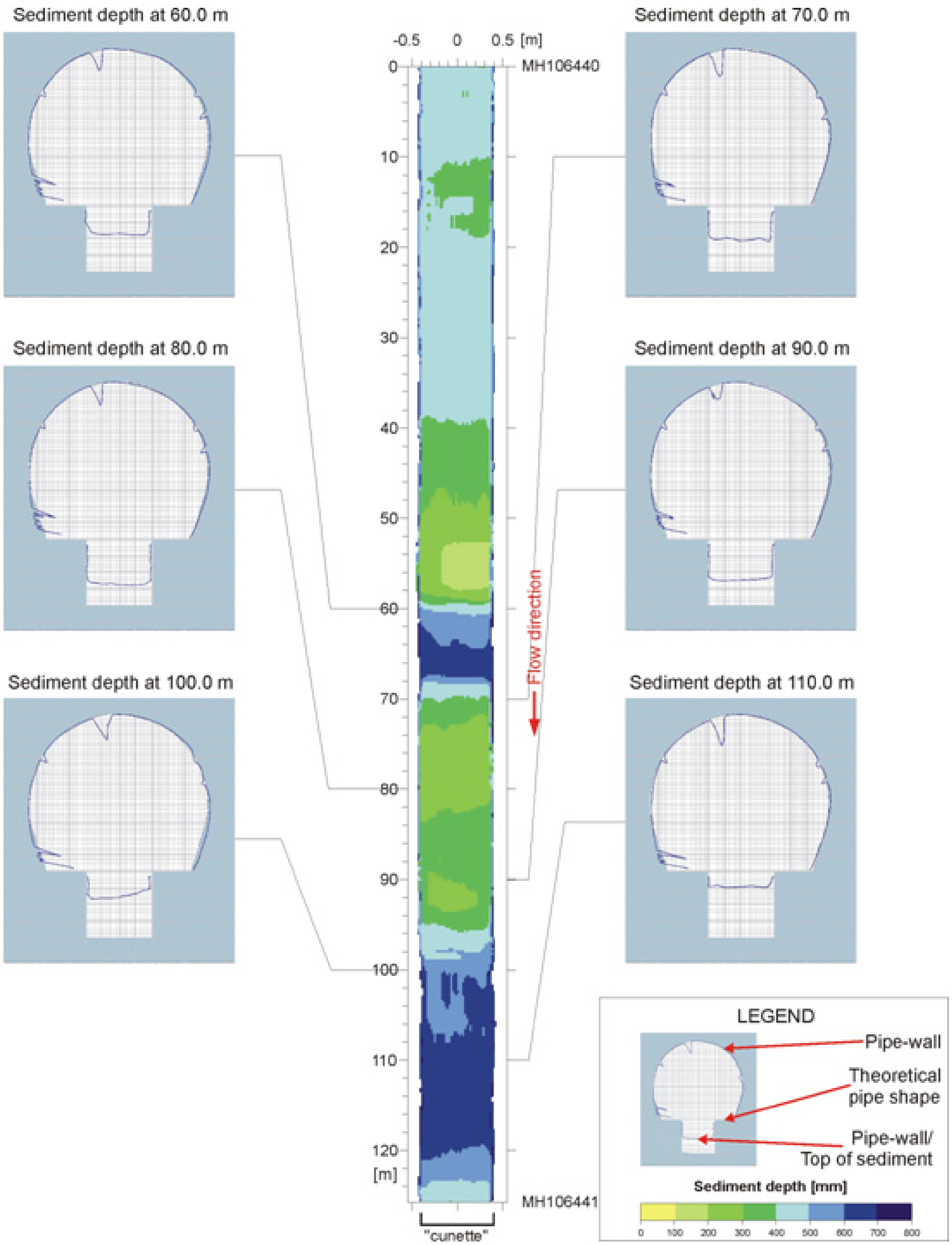

Figure 5. Sediment depth, distribution and cross sections for a 125.8 m section of the Quai Georges Gorse Interceptor.

SUMMARY AND CONCLUSIONS

With limited available funding and budget constraints becoming more prevalent, timing of rehabilitation and overall intelligent asset management is more critical than ever for municipalities and asset owners. Advanced pipe condition assessment technologies, including the SewerVUE multi-sensor pipe inspection system (MPIS) have demonstrated to be cost-effective, non-destructive methods that are able to help better refine structural condition and estimated remaining life of an interceptor, accurately determine overall severity of pipe degradation, as well as provide a basis for improved cost allocation and timing of rehabilitation efforts.

ACKNOWLEDGEMENTS

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Application and Laboratory Tests of Stainless Steel Liner for Trenchless Rehabilitation of Water Mains in China

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Abstract

Stainless steel liner, as an emerging method, is being used with increasing frequency for trenchless renovation of damaged water mains in China. In this paper, the application of rehabilitating water mains and laboratory testing of stainless steel liners are described. Practices prove that this new trenchless technology can be effectively and low-costly utilized to renovate damaged water mains and meet the utility owner's requirements. And considering that there have been little studies on the buckling performance due to external pressure of this new thin-wall structure, the buckling strength of stainless steel liner is studied by laboratory tests. The research shows that the buckling resistance of the DR445 liner is more than 7.25 psi. This conclusion demonstrates that the stainless steel liner meets the requirement in Chinese national standard GB 50332-2002.
Keywords: Stainless steel liner; Supply pipes; Trenchless rehabilitation; Buckling strength.

INTRODUCTION

Using a stainless steel liner for the trenchless rehabilitation of water supply pipes is an emerging approach providing a low cost and a trenchless rehabilitation of water supply pipes in China. The stainless steel liner can be welded with the host pipe formed to become a close-fit liner with small annular gap. Generally, due to the limitation of construction technology, stainless steel liners are mainly being applied to renovate supply pipelines which are larger than 31.5 inch diameter at present. The curved stainless plates are manually in-situ welded to be form cylinders inside the host pipes to be rehabilitated. The most commonly used stainless steels for water supply pipe rehabilitation have the designation 06Cr19Ni10 which is a form of Type 304 stainless steel. Since most pipes used to build water distribution systems in China were built decades ago, and research shows that more than 0.16 million miles water pipes were built before the year 2000(Ma and Zhou, 2013). Now there is an increasing concern about the remaining service life of these aging water distribution systems. Due to long term corrosion and stress-induced deterioration and damage, the structural integrity of many old steel or concrete water pipes is such that they need to be rehabilitated immediately. Carrying out such rehabilitation using open-trench construction and full replacement involves a great deal of work, requires a significant amount of time, and is often very costly (Jeyapalan, 2003 and Najafi, 2013). Stainless steel liners have many desirable characteristics for the trenchless rehabilitation of water supply pipes and they have shown excellent performance and a broad market prospect.

BACKGROUND OF THE PROJECT

The City of Weifang in Shandong Province, which was named as "the World Kite Capital" is located 410 kilometers northwest of Beijing. The city has a population of more than 9 million. The Weifang Water Company owns a 47.2 and 55.1 inch diameter concrete pipes, which are the water mains of the downtown built in 1990 and 2000 respectively, and buried under the cities' main road as showed in Figure 1. Both of the length is nearly 1 mile. Considering the fast development of the city's population and industry, the currently designed pressure 14.5 psi is becoming lower and cannot meet the requirement of the city. The owner plans to improve the internal pressure capacity and supply capacity.

The water mains are located in the main road of the city, the traffic is very busy. These factors made traditional methods of water main replacement impossible in terms of cost and customer service. Due to the high tensile strength to resist internal pressure of the stainless steel, it was applied to renovate the concrete water mains. To guarantee continuous water supply while constructing the stainless liners and reduce the cost for installing temporary by-pass pipes, the owner plans to repair the two water mains in sequence.

Figure 1. Location of the buried concrete pipes

STAINLESS STEEL LINERS CONSTRCTION

1 Required material properties of stainless steel

The most commonly used stainless steels for water supply pipe rehabilitation have the designation 06Cr19Ni10 which is a form of Type 304 stainless steel. For higher requirement of corrosion resistance such as high chloride concentration environment, the designation 06Cr17Ni12Mo2 and 022Cr17Ni12Mo2 which is a form of Type 316 and 316L stainless steel respectively can be utilized as ruled in table 1. The mechanic properties should meet the requirement in national standard GB/T 228-2010, *Code for metal material tensile test* as showed in table 2. In this case, the Type 304 stainless steel was chosen because the renovated pipes are water pipes.

Table 1. Choice for different types of stainless steel

Type	chloride concentration/lb/ft^3	Application
06Cr19Ni10 (Type 304)	≤0.012	Water and gas pipes.
06Cr17Ni12Mo2 (Type 316)	≤0.062	Pipes with higher corrosion resistance than type304.
022Cr17Ni12Mo2 (Type 316L)	≤0.062	Sea water or medias with high chloride concentration

Table 2. Mechanic properties of stainless steel

Property	Minimum value	Testing code
Tensile/psi	7.54×10^4	GB/T 228-2010
Elongation/%	35	
Yield strength/psi	4.64×10^4	
Area reduction ratio/%	30	

2 Construction procedures of stainless steel liners

The construction procedures of stainless steel liners are quite similar as the other pipe liners. And the process can be showed as follow in Figure 2.

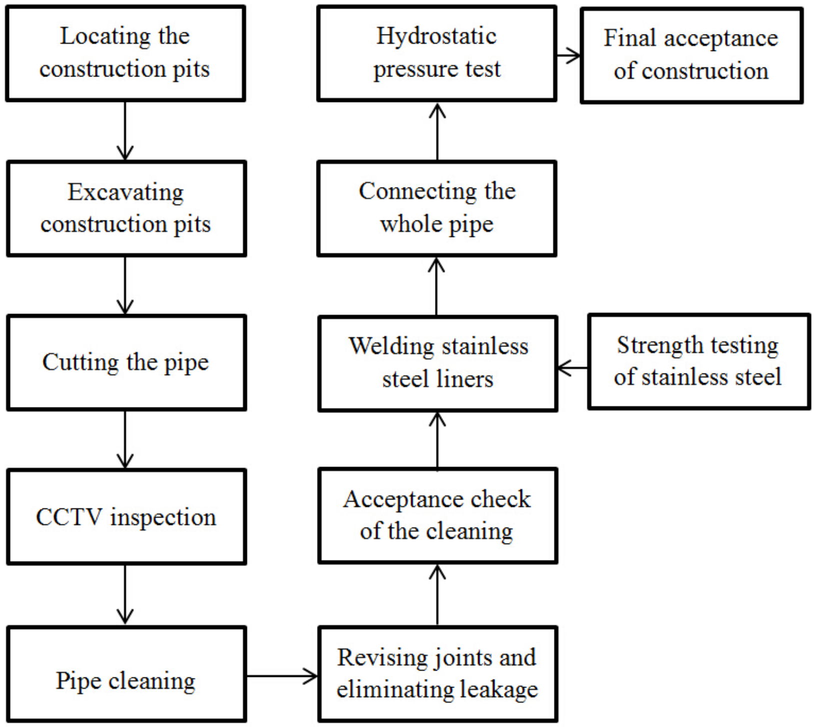


Figure 2. Construction flow-process diagram of stainless steel liners

2.1 Excavation of construction pits

The stainless steel liners are inserted into the water main through the construction pits excavated along the water mains. The location of the pit was selected to minimize the number of pits. To minimize the impact of rehabilitation construction to the surrounded traffic and environments, the area and distance of each construction pit are limited to be less than 19.6 × 9.8 ft and more than 0.87 mile respectively.

2.2 Pipe cleaning

It is a critical step to cleaning the pipe in the rehabilitation of a water main using stainless steel liner. The 15 and 20 years old reinforced concrete pipes are fairly clean. However the deposits and small amount of corrosion on its inside walls have to be removed to allow the stainless steel liners to adjoin tightly to the host pipes and restore its flow capacity. The cleaning tools and procedures are quite the same as done in cured-in-place pipe (CIPP) practice. Cleaning was achieved with the use of a rotary chain boring tool. Using water pressure, the chains rotate inside the pipe knocking off the deposits. After cleaning, the pipe was inspected with a CCTV camera to verify that the corrosions and deposits are removed.

2.3 Joints and leakage treatment

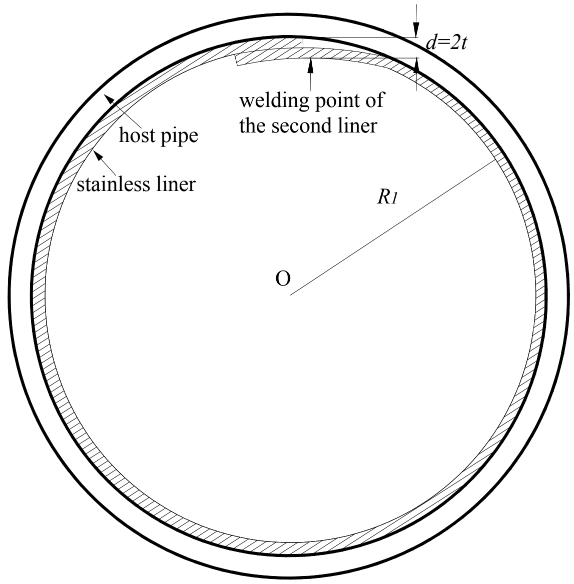
Considering the fact that the host pipes are concrete pipes, any dislocation of the joints and leakage spots will result in gaps between the liner and host pipe. They should be treated by grouting mortar to keep the inner wall of host pipe smooth. The stainless steel liners can adjoin to the host pipes tightly and get enhanced by the host pipe.

2.4 Welding process of stainless steel liners

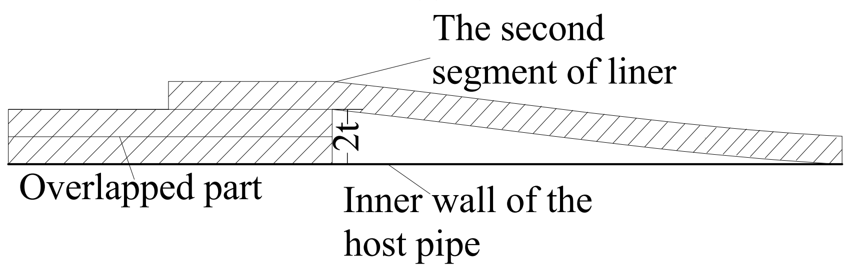
The 7.9 ft long and 0.07 inch thick stainless steel plates are pre-produced in factory, and each plate is curved as showed in Figure 3 by a rounder. The required separation distance of welded joint between each stainless plate is no less than 0.79 inch. And the position of 4 o'clock and 8 o'clock are recommended as welding joints for the adjacent stainless steel plates.

Figure 3. The curved stainless plates

They are manually welded in-situ to form cylinders inside the host pipes to be rehabilitated. Generally, stainless steel liners can be fitted to the host pipes evenly and with a good form-fitting shape using this method. For overlapped and manually welded liners, there would be an inevitable gap adjacent to the overlap point. The first segment of the overlapped liner will cause a gap which is equal to the thickness of the liner, and the second part of liner is welded onto the inner wall of the first liner. And the gap of the second liner near the overlapped part doubles, i.e. d=2t, as shown in Figure 4. The welding quality inspection should be conducted after the welding construction based on Chinese national standard GB50235-97, *Code for construction and acceptance of industrial metal pipeline engineering*. Figure 5 shows the stainless steel liners after insertion into the host pipe in practice by in-situ manual welding method.



(a)



(b)

Figure 4. Gap caused by overlapping stainless steel liners

Figure 5. Stainless steel liners after insertion into the host pipe (from BAODING JINDI SCIENCE&TECHNOLOGY DEVELOPMENT CO., LTD)

QUALITY INSPECTION AND TESTS

1 Welding quality inspection

The welding quality assessment was conducted by tensile and bending strength tests of the welded stainless steel samples. And the conclusion can be made that the tensile strength of the welding joints is higher than that of the stainless steel material. Most of the fracture spots are located beyond the center of the welding joints. And no flaw was found in the surface after being curved in 180° . The inspection proved that the welding quality was excellent.

2 Mechanic property tests

The required material properties including tensile strength, yield strength and elongation at break were tested based on related national standard GB/T3280-2007, *Cold rolled stainless steel plate, sheet and strip*, and the results were compared with the specified values as showed in table 3. The mechanic property tests showed that the applied stainless steel meet the requirement of the national code.

Table 3. Comparison between the Test results of stainless steel samples and specified values in GB/T3280-2007

Property	Tested results of stainless steel samples	specified values in the standard
Tensile/psi	1.02×10^5	$\geqslant 7.47\times10^4$
Yield strength/psi	4.13×10^4	$\geqslant 2.97\times10^4$
Elongation/%	63	$\geqslant 63$

3 Hydrostatic pressure test of the stainless steel liner

As specified a hydrostatic pressure test of independent 47.2 inch diameter stainless steel liner was conducted to investigate the structural resistance to internal pressure. The test was successfully carried out at 166.8 psi pressure for 72 hours, which was more than twice the 72.5 psi operation pressure. And the hydrostatic pressure test of the renovated pipe also was carried out for the final acceptance of construction. The test was conducted at 116 psi for more 15 minutes, and the pressure did not drop which showed a good performance of internal pressure resistance.

BUCKLING TEST OF STAINLESS STEEL LINER

In water pipelines, it is the high tensile strength to resist internal pressure of the stainless steel that needs to be first considered. However, little attention has been paid to the buckling strength of this thin stainless steel liner, whose DRs typically are more than 300 in practices (Ma, 2014). And also the bulges inevitably caused by construction technology will badly impact the buckling strength (Sawy and Moore, 1998). The motivation for vacuum pressure testing is to better understand the critical buckling pressure for the types of stainless steel liners described above. Vacuum buckling testing is a practical way to try to acquire the necessary data to create a guide for the practical use of such liners. Earlier studies showed a length of 10 times its inside diameter (L/D =10) is adequate for representing the condition of a relatively long pipeline in the laboratory, and eliminates the effect of the restrained ends of the liner on the measured buckling pressure (Bakeer, 1999).The length of each test section L is set to be 32.8 ft, and the diameter of tested liners D is less than 2.62 ft. The ratio L/D＞10, which will certainly eliminate the effect of the restrained ends.

For the vacuum testing, each end of the liner is sealed with plates to allow the creation of the vacuum within the test section. Figure 6 shows a schematic of the experimental setup.

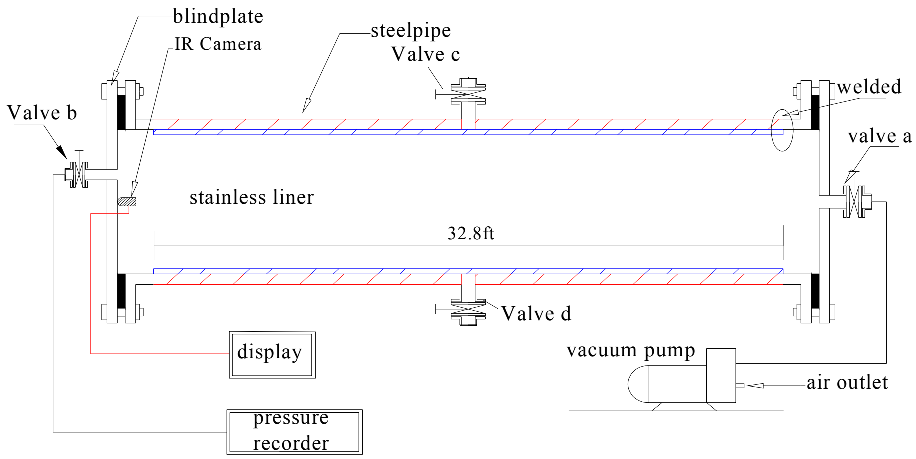


Figure 6. Schematic for experimental liner testing

After the stainless steel liners were inserted into the host pipe, they were welded together to be a whole pipe inside the 2.62 ft diameter pipe. And the conclusion can be made that the quality of the liner insertion for the laboratory tests was equivalent to that typically seen in the field. For the test itself, a vacuum pump is used to decrease the internal pressure of the liner at the rate of 0.725 psi/min with the inlet valves opened. The test lasted for less than 10 minutes. Infrared radiation (IR) cameras were used to record the liner deformation and the buckling failure process. The pressure recorder was used to read the values of buckling pressure. The 0.07 inch thick, diameter ratio (DR) 445 stainless steel liner was tested. Figure 7 shows the initial overall buckling of a stainless steel liner of the test. It can be seen that the buckling is overall along the axis formed at the place where there were obvious imperfections, i.e. the gap due to overlapping of the welding points. And the tested critical buckling strength of the liner is more than 7.25 psi, which means that the stainless steel liner meets the requirement in criterion in national standard GB50332-2002, *Structural design code for pipelines of water supply and waste water engineering*.

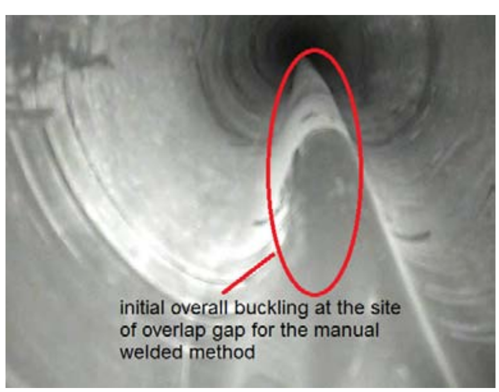


Figure 7. Initial buckling of the liner from IR camera

CONCLUSIONS

Stainless steel liners are an emerging and innovative method used in the trenchless rehabilitation of water supply pipes in China. They have been shown to provide an excellent performance with good installation practices in China. The application of the stainless steel liner showed that the internal pressure resistance of this liner is excellent due to the high tensile strength. And the constructing technology of stainless steel liner will inevitably cause a gap which is twice the thickness of the liner. This paper provides an initial investigation of the critical buckling strength of stainless steel liners inserted into host pipes by conducting full scale laboratory buckling tests. And the test showed that the DR445 stainless steel liner have a buckling strength of more than 7.25 psi after the insertion into a steel pipe. The conclusion shows that this innovative method for trenchless rehabilitation of supply pipelines meets the requirement in the relative Chinese national standard. And more detailed research on the buckling strength and the design theory of stainless steel liners is needed in the future.

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Non-Invasive and Remote Pipeline Rehabilitation Technology Using Reactive and Magnetic Particles

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Abstract

We propose a novel pipeline rehabilitation technique that uses particles of a reactive, multi-layer metallic foil to repair internal cracks of a pipeline with minimum on-site effort and no downtime. The principle of this repair technique is as follows: when cracks are detected during routine pipeline maintenance, the particles are introduced into the fluid flow in the pipe, then manipulated by an external magnetic field to fill the cracks or pits. Once the particles are in the site of interest, induction heating is externally applied, causing the reactive metallic foil to undergo an exothermic diffusion reaction and sintering the particles in the crack. In this paper, we experimentally confirm the feasibility of such a reaction within a pipe repair system. We investigate the reaction and bonding strength for various simulated crack sizes, particle sizes, and particle mixture compositions. We observed stable reaction propagation in 30 mm long, 100 µm diameter glass capillaries at velocities of 80-100 mm/s. Shear tests were also performed on the reacted particles. A maximum shear stress of 3 MPa was applied between sintered particles and a simulated crack in carbon steel, demonstrating the ability of reactive particles to repair small pipe cracks.

INTRODUCTION

Pipelines that transport petroleum product, natural oil, or water are our life lines. Inadequate maintenance of these pipes poses the risk of leaks, ruptures, or explosions, leading to property damage and environmental destruction. Modern technology allows early detection of damage to a pipe through the use of non-destructive inspection tools. However, it is economically difficult to repair these damages when they are detected (Castanier, B. et al, 2006), especially for pipelines in remote locations, such as off-shore pipes. Several repair technologies have been proposed for remote locations, such as mechanical clamping of off-shore pipes (Espiner, R. et al (2008)), recoating the inside of the pipe, mechanically sealing the pipe, or inserting a new pipe inside an existing pipe (Morrison, R. et al, 2013)). All of these technologies require major on-site efforts (e.g., heavy construction) and also force the suspension of the pipeline's operation, causing lost revenue while the pipe is repaired.

There is strong demand for non-invasive technologies that can repair minor pipe damage with little on-site effort. We introduce a technique for filling pipe cracks by using a particle mixture of solder and a reactive, multi-layer metallic foil. The particle mixture is injected in the fluid flow and guided to the crack via an externally applied magnetic field. The application of an energy source (e.g., induction heating, electrical current) starts a large exothermic reaction in the foil, which sinters the solder and repairs the crack. As the particle mixture is introduced to the pipe during its operation, no revenue losses result from taking the pipe out of operation. Additionally, this method does not require precise alignment of repair equipment with the crack, reducing the on-site effort for maintenance.

In this paper, we determine the optimal composition and size of the particle mixture for pipeline crack repair. We also experimentally confirm the feasibility of this technique through reaction propagation and bonding strength tests.

NON-INVASIVE REPAIR USING THE PARTICLE MIXTURE

Figure 1 shows the principles of the repair technique. Once the damaged areas are identified, particles are introduced and suspended in the fluid stream. Mukherjee, D. et al. (2014) previously introduced a method to guide particles into position using a longitudinal magnetic field on the pipe wall. The pipe discontinuity at the crack site

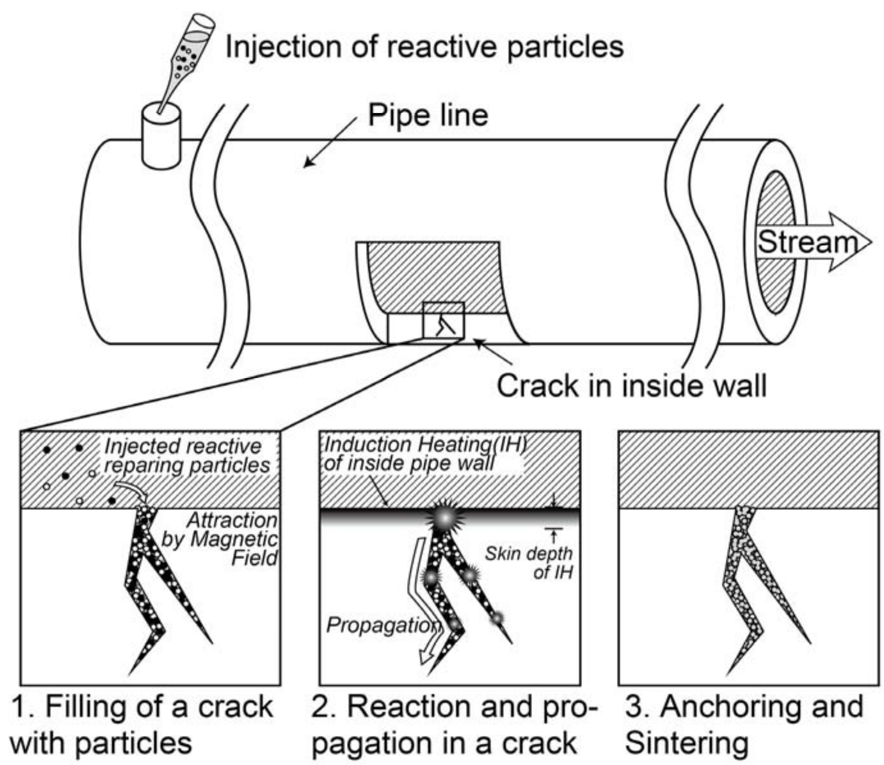


Figure 1. An overview of the three major steps in the non-invasive pipe repair technique.

causes leakage of magnetic flux and preferentially attracts the particles. This paper does not explore particle guidance within the pipe, but the method is briefly described to illustrate a complete repair system.

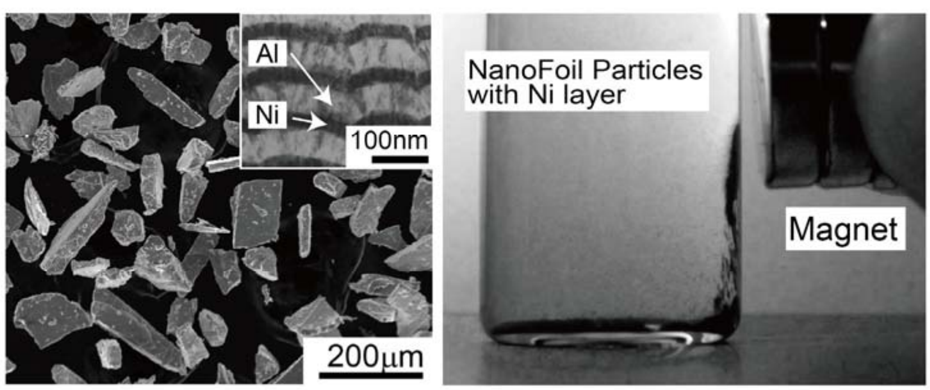
After particle guidance into the crack, the exothermic reaction of the reactive foil particles is activated through an external source, such as induction heating. Once the reaction begins, the surrounding material is heated, sintering the particles in the crack. Multiple damage sites may be repaired in parallel by applying the activation energy source to several cracks in the same area.

COMPOSITION AND FABRICATION OF PARTICLES

The particles used in this technique are fabricated by modifying 40 μm thick NanoFoil (NF40, Indium Corp., Clinton, USA), a sheet of reactive foil composed of alternating nanoscale layers of Al and Ni (inset, Figure 2). Nanofoil undergoes a self-sustaining exothermic reaction when it is thermally or physically shocked (Indium Corporation, (2012)). As the reaction is not combustion-based and does not require oxygen to proceed, the reaction could take place in a sealed pipe.

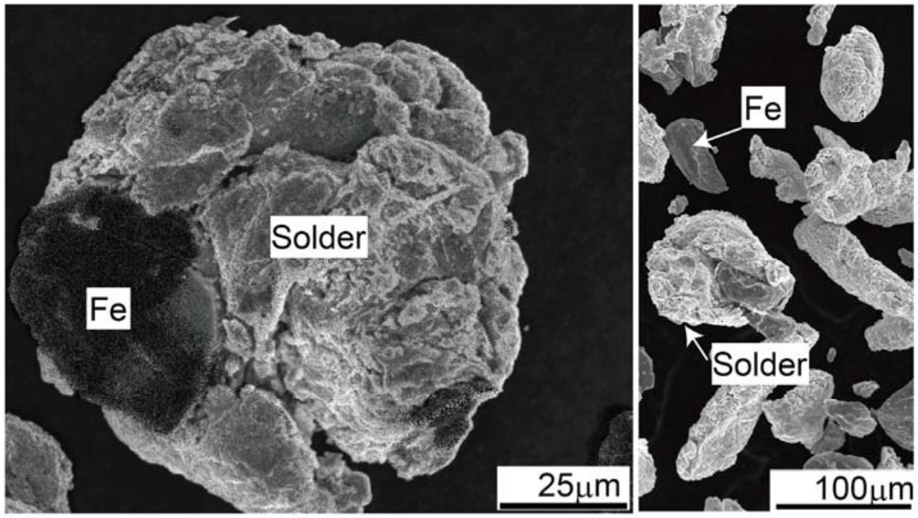
The NanoFoil particles are prepared by mechanical grinding using a mortar. The NanoFoil sheets are covered by water or ethanol during grinding to prevent a premature reaction due to the physical shock of grinding. The reacted product of the reactive foil does not take a liquid phase, so a low melting point solder, $Sn_{96}Ag_4$ (ASTM96TS), is introduced as part of the particle mixture to improve the crack conformality. The Sn alloy solder is ground to size using a file. A scanning electron micrograph of the solder with energy-dispersive X-ray spectroscopy is shown in Figure 3.

To increase the mobility of the particles in the externally-applied magnetic field, each sheet of foil may be electroplated with a 1 μm thick layer of Ni just prior to grinding as demonstrated in Figure 2(b). Similarly, the solder particles are mixed with Fe particles in the mortar to enhance their magnetic properties.



(a) (b)

Figure 2. (a) A scanning electron micrograph of several reactive foil particles (inset: cross-section of the NanoFoil). (b) A photograph demonstrating the motion of the NanoFoil particles in a magnetic field.



(a) (b)

Figure 3 Scanning electron micrographs of (a) one solder particle shown with energy-dispersive X-ray spectroscopy false color, and (b) several solder particles

To sort the particles by size, the solvent is evaporated and the particle mixtures are sieved. Solder-reactive foil particle mixtures are separated into four average particle diameter ranges: <45 µm, 45–65 µm, 65 µm–90 µm, and 90–200 µm. In the following sections, the solder-reactive foil concentration of these mixtures is given as a volume percentage of the solder.

RESULTS AND DISCUSSION

Reaction Speed in Glass Capillaries

Glass capillaries were used as simulated cracks to investigate the reaction speed, and to determine if the reaction is quenched as the crack size decreases. First, we investigated the reaction speed under various capillary tube diameters and particle sizes by photographing the reaction with a high speed camera. A quantity of NanoFoil particles without the filler metal was loaded into a glass capillary and activated using a pulse of electric current. Figure 4 shows snapshots of the reaction in glass capillaries with various inside diameter (ID).

The reaction speeds as measured by the high-speed camera footage (~100 mm/s) were much lower than those of a continuous NanoFoil sheet (2–10 m/s). The NanoFoil particle reaction is self-propagating up to a distance of a few cm, even with a 100 µm capillary, where quenching effects should be most prominent. To dissociate influence of capillary size from the experiment shown in Figure 4, The relation between reaction speed and particle size was investigated under same packing density and same diameter size of the capillary. As shown in Figure.5, the reaction speed increases as the particle size decreases. This finding suggests that the packing factor is critical to the reaction speed, however crack size is less so.

The solder concentration in the particle mixture also plays a role in reaction propagation velocity. Figure 5 shows the relation between reaction speed and the

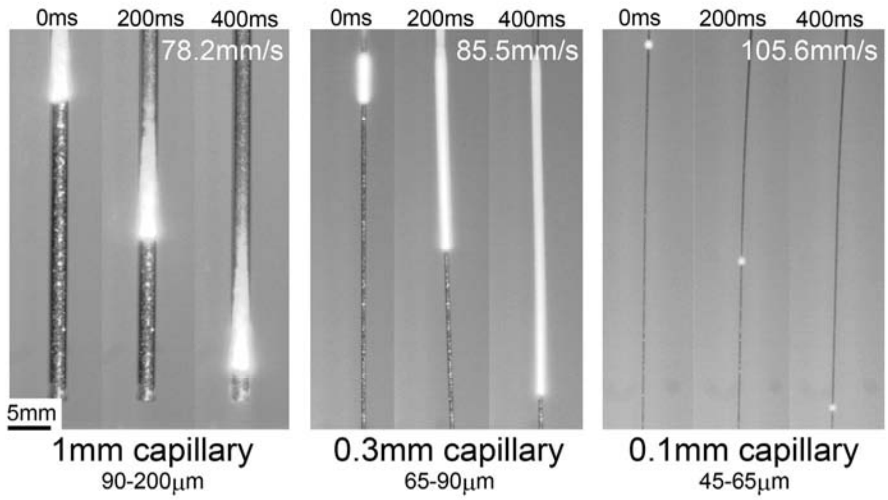


Figure 4. High-speed photographs demonstrating the reaction of NanoFoil particles in a glass capillary with various capillary diameter and average particle size.

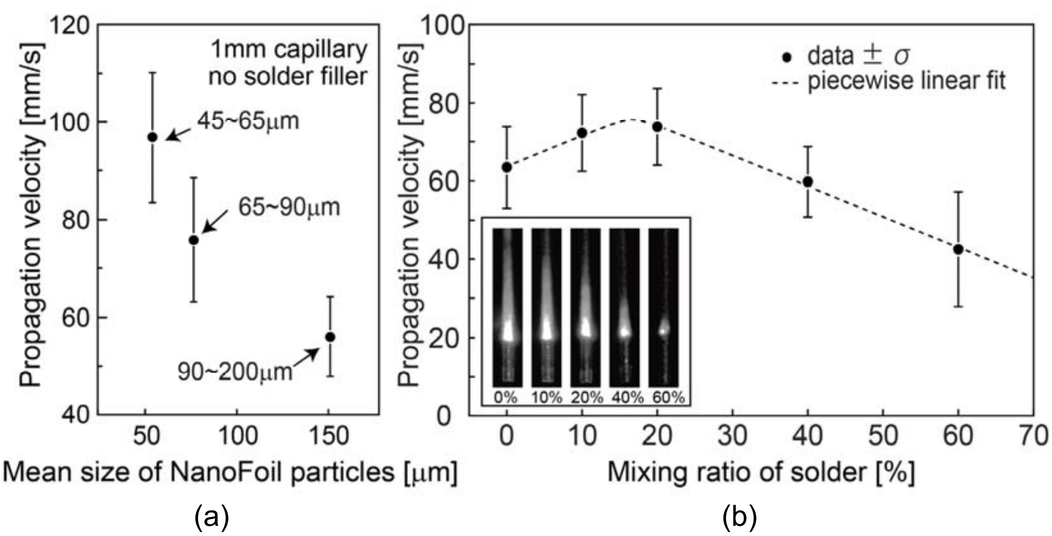


Figure 5. Change of combustion speed in capillary due to particle size (a) and mixing ratio of solder (b).

concentration of the solder observed in a 1 mm diameter capillary. In this reaction, 90~200um NanoFoil and solder particles are used. The maximum reaction speed occurs at a 20% solder particle concentration. We believe that this peak indicates the trade-off between increasing the thermal conductivity due to melting of the solder and decreasing the energy density of the mixture. The upper limit of solder concentration occurs at 60%, and is also the lowest reaction rate we measured. Above this concentration of solder particles, the reaction does not reliably self-propagate.

Repair of a Carbon Steel Crack

The repairing performance of this technique is evaluated by reacting the particles in a simulation crack made of a cylindrical hole (1 mm in diameter) in a carbon steel block (ASTM A516). Figure 6 shows the experimental setup. The test blocks are prepared by drilling a carbon steel block with a spiral reamer to get a smooth inside surface. The test blocks are single-use and are cleaned by ultrasonication using isopropyl alcohol before the experiment. These Nanofoil sheets were not plated with Ni before grinding. The particle mixture is loaded into the hole with a plug at the bottom (Figure 6(2)). After activation with an electrical current, the force required to push out the sintered particle mixture is measured with an FC22 load cell (Measurement Specialities, Hampton, USA). The shear bonding strength is calculated by dividing the maximum force by the surface area of the test hole (9.4 mm^2).

Figure 7 is a plot of the shear bonding strength for various solder concentrations. The bonding strength rises with increasing solder concentration until around 40% solder, and then falls due to decrease in energy concentration. The peak of bonding strength is measured as 0.3 MPa at 40% solder, as shown in Figure 7(a). The effect of solder flux on the bonding strength was also studied. In these experiments, NOKORODE regular paste flux (Rectorseal, Houston, USA) is used to increase the bonding strength by increasing the solder wettability to carbon steel. The improvement of wettability was confirmed by measuring contact angle of melted solder on carbon steel (148° without flux, 44° with flux). The flux is applied by inserting a flux-coated rod into the hole. The plate is heated to 100℃ to flow the solder, the particles are introduced at room temperature. Once the solder flux is introduced, the maximum bonding strength increases by a factor of 10 in the optimal particle mixture (Figure 7(b)). The solder flux may enable better thermal transport and improves contact between the particles and the sidewall.

The reacted particle mass is inspected by scanning electron microscopy. As shown in Figure 8, an energy-dispersive X-ray spectroscopy image indicates the NanoFoil and solder particles are sintered. Generally, the NanoFoil particles hold their shape while the surrounding solder appear to coat the NanoFoil particles. Furthermore, some carbon steel debris in the form of Fe is present on the surface of the sintered particle mass, suggesting that bonding likely occurred between the simulated crack and the and particle mixture.

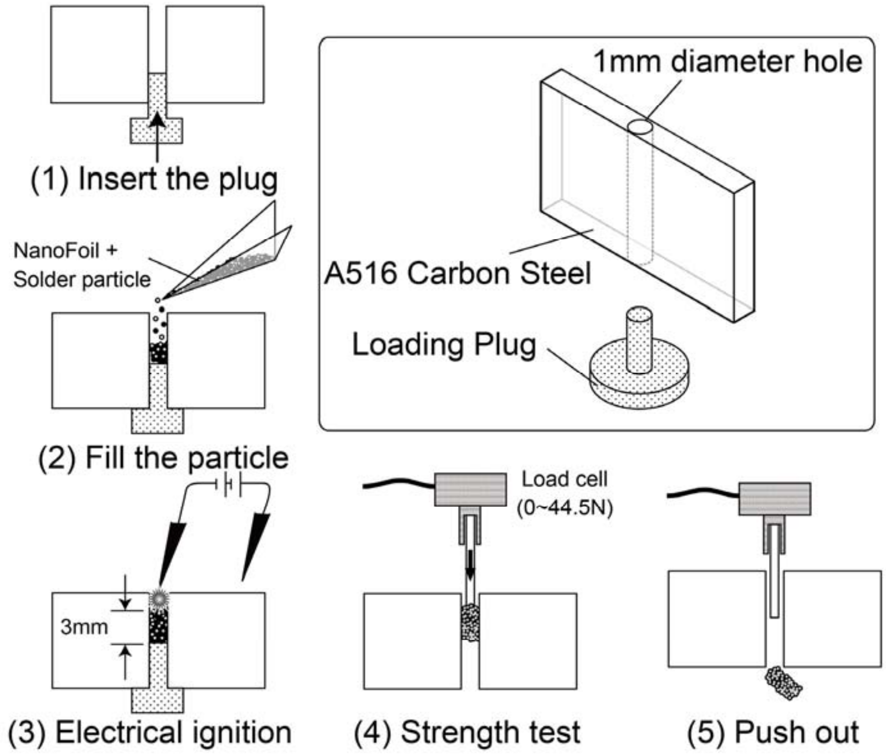
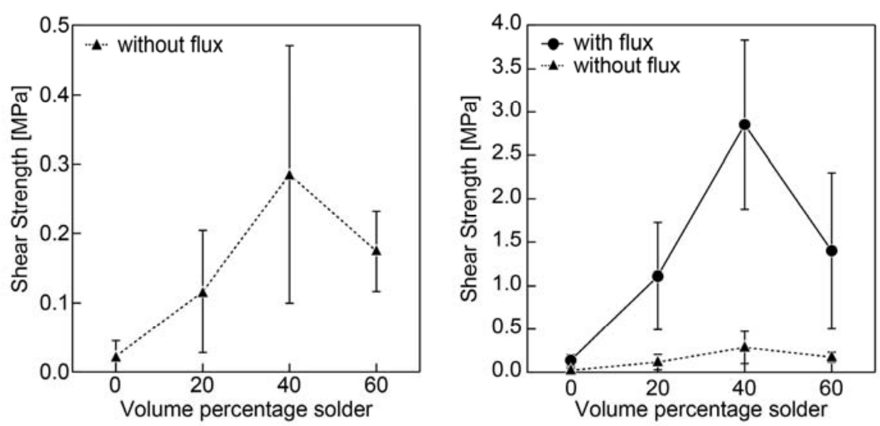


Figure 6. Procedure of the experiment for shear strength measurement



(a) (b)

Figure 7. Plots of the shear strength vs. mixing ratio (a) without and (b) with a solder flux coating.

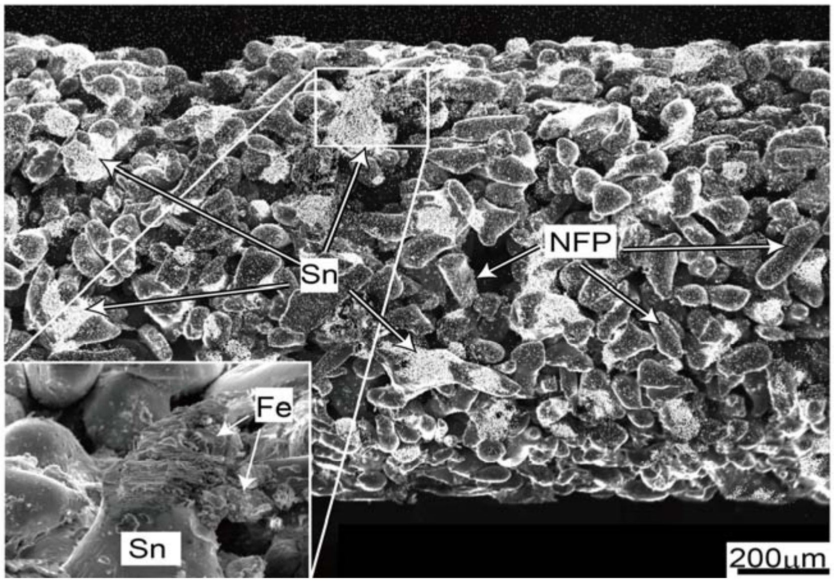


Figure 8. Scanning electron microscopy image with false color energy-dispersive X-ray spectroscopy (mixing ratio: 20%, no solder flux)

CONCLUSION

In this paper, we presented a non-invasive repairing technology designed to repair a pipe without suspension of operation and with less on-site effort. We synthesized an energetic and magnetic particle mixture and measured its reaction speed in simulated cracks. The reliability of this technology is also experimentally estimated by measuring the shear strength between sintered particles and carbon steel. The strongest bond (3 MPa max shear) occurred at a mixture composition of 40% solder/60% NanoFoil particles, which balances the use of solder as a crack fill material while leaving enough reactive NanoFoil to propagate the reaction.

As suggested by Figures 5 and 7, the concentration of solder is a major factor in bonding strength, while the propagation speed is governed by the particle size and mixture composition. Coating the simulated crack sidewalls with solder flux further increased bonding strength. In future work, we will add microcapsules of flux into the particle mixture to measure the effects on bond strength.

In conclusion, a preliminary investigation toward novel pipe repairing technology is completed. These findings will be helpful for more advanced development of pipeline repair technologies that use reactive particle mixtures, enabling improved pipeline maintenance without increasing cost.

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Engineering Rehabilitations Based on Non-Destructive Examinations

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Abstract

While a combination of condition assessment and pipeline rehabilitation is used extensively for wastewater sewer mains, this is not true for water mains. One reason is the greater difficulty of assessing water mains than sewer mains (a simple video inspection does not suffice). Another reason is the lack of guidelines for selecting and designing a rehabilitation system which considers the condition of the old water main. This paper summarizes recently completed Water Research Foundation Project 4473, "The Assess-and-Fix Approach: Using NDE to Help Select Pipe Renewal Methods". This report makes the case for employing condition assessment as part of rehabilitation projects, selecting and designing the final lining only after first scanning the pipe for defects. To develop useful guidelines, the study investigated the essential properties for structural lining systems, and how various linings might be applied to impaired mains. Application of the assess-and-fix approach is feasible today, but requires knowledge and diligence on the part of utilities. Guidance is now available through this recently completed study. Utilities are encouraged to adopt this approach.

INTRODUCTION: WHY USE NDE FOR SMALL WATER MAIN REHABILITATION

Few water utilities currently employ active condition assessment methods for small diameter water mains. Renewal decisions for these assets are generally based on break frequency. After a main has been repaired several times, its condition is deemed to be poor, and a new main is planned. Although the planning for main replacement can be quite sophisticated taking into consideration many factors, the actual evaluation of pipe condition is normally based largely on leak and break repair records. This repair-on-failure approach is acceptable because a break on a small main is generally not a high-consequence event, and many managers would rather spend money replacing pipe than assessing it.

Even though break frequency is usually the prime criterion for renewing a main, it is not uncommon for mains to be replaced merely on the basis of age. Although it is well understood that age is a poor predictor of pipe condition, utility managers will sometimes elect to replace a main because the perceived risks associated with the main are judged too high. The decision to replace an old main without a history of breaks is also often driven by other factors—concurrent work along the street, or the general idea that infrastructure must be renewed in a timely manner.

Contrast these management strategies with wastewater systems. Although wastewater and water pipes are frequently managed by the same utilities, decisions about renewal are quite different. For wastewater, few would argue that a whole pipe is worthless because one or two repairs were needed. For wastewater, no one would decide to replace a pipe merely because of its age. For wastewater pipe, condition assessment is routine and is the driver for nearly all condition-based renewal decisions. Why is this? Because the method (video inspection) is inexpensive, easy to deploy, and results are readily understood. Not only does the condition assessment method produce a picture (which even a layperson can understand), but the industry has developed standard inspection protocols and defect codes for documenting the results.

The Water Research Foundation has funded two projects which aim to broaden the acceptance of in-pipe non-destructive examinations (NDE) for small diameter water mains. Project 4471 proposes to use NDE in a relatively non-disruptive manner to "sample" pipe in a system, then apply the information to infer the conditions of similar pipes. Project 4473 goes a step further, proposing to combine the assessment, engineering, and rehabilitation of water mains into a single product delivery (the "Assess-and-Fix Approach"). This latter project was recently completed and its final report will be published this year (2015).

This is the third paper presented to ASCE Pipelines regarding the Assess-and-Fix study. In 2013, the need for the project and basic concepts were outlined. In 2014, progress was reported, including laboratory testing of rehabilitation methods and field tests of NDE inspection methods. This paper discusses the results of the study, providing guidelines for how a rehabilitation method can be selected and designed using data from detailed NDE scanning of the pipe.

THE ASSESS-AND-FIX CONCEPT

So why are managers unwilling to assess water mains? The technology has existed for nearly 20 years and is well proven. The arguments against using the technology are:

1. **It's too expensive**. Getting a tool inside a water pipe takes too much effort, and the assessment services cost too much.
2. **It's too risky**. The tool could get stuck or contaminate the water.
3. **It's too confusing**. The condition assessment may produce hard-to-interpret results.

Similar arguments are heard regarding water main rehabilitation and why similar long-established technologies are not more broadly used:

1. **These pipes are too old.** How do I know the final product will last?
2. **It's not cost effective.** We tried rehab once, and didn't save much money. We would rather invest in new pipe, where the life-expectancy is better understood.
3. **It's unproven.** We've tried several methods, but have not adopted any. Where are the standards?

By adopting an assess-and-fix method for water main renewal, and by implementing it on a large scale, a water utility could achieve results similar to how wastewater mains are managed. Bad sections of pipe would be rehabilitated, good sections of pipe would be left intact, and the method of rehab would be appropriate to the defects. Most importantly, rehabilitation rather than open-trench would be employed, the street would not be torn apart and projects would be completed more quickly.

The assess-and-fix concept is to perform condition assessment at the same time that a pipe rehabilitation project is underway. The decision to rehabilitate a pipe may be made using various factors, as is currently practiced, but the method of rehabilitation is not selected until the pipe is scanned and its condition is determined. Unlined cast iron pipes are classic candidates for an assess-and-fix approach, but so is any pipe whose condition is believed to be compromised and where a trenchless method of renewal would be beneficial.

The difficulties and risks associated with scanning a water main are eliminated, if the inspection is performed as part of a cleaning and lining rehabilitation project. In rehabilitation projects, temporary bypass water systems are first installed, and holes are excavated to gain access to the pipe. The pipe is then cleaned using mechanical scrapers pulled through the pipe. The final step is to line the pipe. If scanning is performed after the cleaning, but before the lining, the added field effort is minor. The scanning tool can be pulled through the pipe at the same time that a final video inspection is often performed. The NDE data can then be evaluated and a lining selected and designed. On a project involving multiple mains, crews could be directed to other work, while the engineering evaluation is completed. In this way, work progresses without significant delay to the project and impact to crew inefficiency. An assessment during rehab is thus very manageable.

Making the appropriate lining adjustments should also be manageable. When a spray-applied polymer lining is being used, the thickness of the lining is increased or decreased by adjusting the travel speed of the sprayer. With an appropriate contract mechanism, an owner can go from a non-structural to a semi-structural lining, by agreeing to pay for a thicker lining. If the evaluation indicates the need for a fully-structural method, the contractor may need to procure materials for a cured-in-place pipe (CIPP) lining or a pipe bursting application. This could delay completion of a main by several days. In the meantime, the access holes would be traffic-plated while work continues elsewhere. If the project is large enough, a wide range of lining choices should be feasible without significant overall disruption to the schedule, but good up-front planning would be necessary. An owner could also facilitate these adjustments and mitigate delays by paying to keep lining materials on hand. Materials that are not used on one project will find application on another, particularly if the infrastructure program is large and continuous.

While committing a pipe to rehabilitation before it is assessed is counterintuitive, it's not really that crazy. Miles of unlined (pre-1940) cast-iron pipe are still found in many systems. Rehabilitation of these mains can be justified by the water quality and hydraulic benefits achieved by lining, not to mention the life-extension attained by eliminating internal corrosion. Many utilities, large and small, already do this, and have been for decades. Through long-running programs of rehabilitation, several large utilities have in fact completely eliminated unlined cast iron pipes from their systems. The assess-and-fix approach merely advocates deferring final selection of the rehabilitation method until the pipe has been scanned, and its condition is known. At utilities that have implemented large-scale lining programs, the cost per foot of pipe accomplished ranges from 20 to 60 percent of the cost of replacement. The added cost of NDE scanning should not significantly alter this cost advantage, while promising the added benefit of a longer-lasting, better-defined product.

Similarly, utilities often commit to replacing mains based on leak history, age, and other factors. If these utilities were to commit to trenchless renewal as their primary method of main replacement, an assess-and-fix evaluation could be used to optimize these renewals. A few utilities already use trenchless methods as their primary means of infrastructure renewal. By adding assess-and-fix evaluations to their procedures, renewals could be custom-tailored to fit the true conditions of the mains. In many cases, less expensive rehabilitation methods could be employed. Arguably, there is little that is accomplished through open-trench replacement of small diameter mains that cannot be accomplished just as well with a low-impact trenchless method. This is particularly true through an assess-and-fix approach that matches the rehabilitation to the condition of the main.

APPROACH TO DEVELOPMENT OF ASSESS-AND-FIX GUIDELINES

An engineering approach was employed to develop the assess-and-fix guidelines found in this the study's report. These guidelines expand upon existing well-established standards and manuals of practice, while applying the latest research and basic engineering principles. In some cases, assumptions were made where knowledge gaps existed. Because the guidelines are intended for small diameter water mains (12 inches and smaller), a perfect methodology is not needed. Small water mains are "low-consequence" assets. They are allowed to fail occasionally. By accepting the possibility of such occasional failures, over-conservatism is avoided, and greater overall economy is achieved. Also, simplicity is favored. A guideline that is overly complex will not allow for the timely field decisions needed for assess-and-fix rehabilitation, and will never be widely adopted.

By necessity, these guidelines attempt to tie together several "loose ends"—issues that are not fully debated (much less resolved) within the industry. For instance, what are the basic requirements of "fully structural" or "semi-structural" lining systems? Where should different lining systems be applied? How should lining systems be designed? And most importantly, how can the likelihood of a future rupture be determined from NDE data? While partial answers can be gleaned from various sources, this report synthesizes and expands upon the available answers, providing guidance for assess-and-fix rehabilitation that can be implemented today.

Fundamental Requirements for Structural Lining Systems

This study clarifies several important criteria for structural linings, including the paradoxical properties of adhesion and tear resistance. In determining the structural value of a lining system, a primary consideration is whether a lining has the ability to withstand the cracking of the host pipe. If a lining does not keep the water inside when the host pipe cracks, it cannot be considered fully-structural, and its value as a semi-structural lining is also greatly diminished. While AWWA Manual M28, "Rehabilitation of Water Mains" alludes to this requirement, the criteria for a structural lining system are far from clear. As a result, lining systems have been advertised as fully structural, when in fact, they are not. Basic material mechanics indicates that linings which adhere are likely to tear when a host pipe cracks, even if the crack is small. This means adhesion of the lining to the host pipe is undesirable if cracking of the pipe is likely.

On the other hand, good lining adhesion can also be a good thing. Tight adhesion to the host pipe is often needed to connect the lining system to the service laterals, and a good connection between lining and lateral is necessary if the lining is to have structural value. Without adhesion, a more difficult mechanical connection between the lining and lateral is required. Adding these mechanical connections increases cost and often involves digging holes at many service connections, reducing the benefit of "trenchless" construction. This means adhesion of the lining to the host pipe is desirable, if digging is to be minimized.

So the question becomes, to adhere or not adhere? There are advantages and disadvantages. Spray-applied linings are the easiest and least expensive, but are likely to tear upon pipe fracture. More robust, non-adhered linings are more likely to survive pipe fracture, but may require added effort to connect the lining to the lateral. There is no current system that is both adhering and non-adhering (like a Post-it™ Note). By employing an assess-and-fix analysis, the condition of the pipe is used to decide whether an adhered lining is or is not appropriate.

Tests performed for this study confirmed that spray-applied linings should not be assumed to survive host pipe cracking. Even if the adhesion is not good, a frictional bond is created by the internal pressure in the pipe. On the other hand, an earlier, manufacturer-sponsored test has indicated that a CIPP lining may be tear-resistant, but there are questions and issues associated with this test that merit additional investigation. Utilities adopting a large rehabilitation program are encouraged to perform their own tear-resistance testing on real samples of their own in-situ lined pipes. Likewise, utilities are encouraged to verify that linings and laterals are positively connected at service laterals and other discontinuities. This may involve excavating and extracting a few of these connections for examination.

Methods for Assessing Future Pipe Condition

When this study was first conceived, a specific NDE tool was envisioned, which uses remote-field electromagnetic testing (RFT) scanning. This particular tool has been around for nearly 20 years, and has been validated in various independent studies. However, the use of other technologies including newly developed magnetic flux leakage (MFL) scanning tools may also be feasible for assess-and-fix evaluations of iron and steel mains. The basic requirement for iron main assess-and-fix assessment is that the NDE method needs to detect the depth, size and spacing of corrosion pits, and also measure the general thickness of the pipe wall. Technologies that provide a general assessment would not be suitable.

Because water main rehabilitation is intended to last many decades, it is important to design for the future (not current) conditions. It is therefore important to distinguish between the external corrosion pits, which will continue to grow, and the internal pits, whose growth will be arrested once the lining is applied. RFT and MFL tools do not indicate which defects are on the outside or inside of the pipe. To differentiate external from internal corrosion, the NDE scanning should be coupled with in-pipe video inspection (and possibly laser profilometry).

For forecasting external pit growth, a fuzzy-logic model developed through another WRF study is useful. This model shows that pit growth follows a logarithmic curve, slowing substantially as the pipe ages. According to this model, a pit that is 8 mm deep after 75 years should grow by only 1 mm in the next 50 years. Thus, for a relatively old pipe, the future condition will not be dramatically different from the current condition, but pit growth should still be taken into consideration. Applying this model is quite simple; all one needs to know is the current depth of the pit and the age of the pipe.

Selecting a Rehabilitation System to Match Pipe Condition

To select a lining method, a decision tree is provided in the report involving four simple questions:

(1) Is hoop strength significantly impaired? If yes, a fully structural method is needed.

(2) Is bending and axial strength significantly impaired? If so, a tear-resistant liner is needed.
(3) Is significant joint leakage expected? Then a semi-structural method is appropriate.
(4) Is a through-wall hole likely? If yes, a semi-structural method is also appropriate.

The default condition (no significant impairment) warrants a non-structural lining method.[1]

Of these questions, the second question is the most difficult, because no standard exists which defines beam-bending deficiency. Each situation is different. Most mains are not intended to be bent, yet we know from experience that failures from beam bending are very common. If a material is brittle, bending can fail even a main with little deterioration. Circumferential breaks caused by bending and axially loadings are influenced by the soil, traffic loading, variations in temperature, topography and other factors.

The fundamental purpose of the NDE assessment is to determine both the probability of host pipe failure and the modes of failure that are likely to occur. Depending on how a host pipe might fail, different lining designs are warranted. Three different methods of making this assessment are proposed: statistical modeling, deterministic modeling and risk assessment modeling. None of these methods is perfect, but by considering more than one approach and applying good engineering judgment, reasonable results are attainable in a reasonable time frame. Again, analytical perfection should not be a requirement for low-consequence water mains.

Statistical modeling

Desk-top studies of available data are usually the first step in condition assessment. By examining various pipe characteristics (age, material, diameter, pressure, soil type, etc.) and historic records of repairs, the probabilities of different types of pipe failures can be estimated. Statistical analyses are thus important for planning assess-and-fix projects. Mains may be selected for assess-and-fix renewal, based on studies that show a high probability of impairment (structural, water quality or hydraulic).

These statistical analyses also provide valuable input for calibrating the results from the other analyses. For good reasons, engineers are taught to be conservative in their assumptions and analyses. Conservatism saves lives and protects property. But for renewal decisions involving miles of low-consequence assets, over-conservatism can waste money. When looking at pipe condition data, there may be may be a tendency to assume the worst—believing a high likelihood of pipe rupture exists, when the risk may in fact be tolerable. Statistical analyses of break data are useful for ascertaining the true likelihood (the mean and standard deviation) of various occurrences, helping an engineer avoid overly conservative assumptions.

Ideally, a utility will eventually perform enough NDE scanning that statistical relationships between NDE data and break data could be developed. For instance, a utility might know the likelihood of a beam break at the point when pits reach a certain size, in pipes of a certain

[1] AWWA Manual M28 provides guidance regarding lining methods considered fully structural (Class IV), semi-structural (Class II or III) and non-structural (Class I).

vintage, in expansive clayey soils, in a certain part of town. Just as baseball managers use statistics to decide on a change of pitchers, a pipeline manager might use statistics to decide on a change of linings.

Deterministic modeling

Deterministic methods involve applying scientific and engineering principles to predict future conditions and calculate stresses. This is the natural approach for pipeline engineers, because it is how they are taught to design new pipes and other things. They consider the various load cases, the properties of the materials, calculate the stresses, apply safety factors, and are assured their creations will last for many decades.

However deterministic models are fraught with complications that render them difficult to apply to old water mains. These include difficulties in knowing whether a pipe is under a bending load, what exact materials were used in its construction, what defects currently exist (including casting defects and fatigue weakening), and how much additional deterioration will occur. It is common to have little knowledge of the actual wall thickness and the actual mechanical strengths of the pipe, yet this information is necessary for an accurate estimate of pressure and bending stresses. Worst of all, the stress calculations can be quite complex without necessarily producing accurate, reliable results. Varying patterns of corrosion pits create complex 3-dimensional structures that are not easily modeled. False negatives and false positives are both likely to occur with deterministic modeling.

Risk Assessment Modeling

Because decisions need to be made quickly and without significant analytical cost, simplicity is favored for assess-and-fix evaluations. Risk assessment models can be fairly simple. In risk assessment modeling, the relative risks of failure are evaluated based on various factors. Where the likelihood of a particular failure mode is considered high, a lining should be selected that accounts for that failure mode. Where the consequences of failure are also high, a bias towards conservatism (higher factors of safety) is warranted.

The problem with most risk assessments is that they are often subjective and only produce relative risks. One pipe is judged to be riskier than another. So if a pipe is found to be “high-priority”, does this mean it is about to fail, or merely the worst one in a healthy population? To account for this, risk assessment models need to be calibrated. The statistical and deterministic models can provide these calibrations.

Designing Rehabilitation Systems to Match Host Pipe Conditions

The soon-to-be published project report provides recommendations for selecting and designing lining systems, based on evaluations of future host pipe integrity. These recommendations include suggestions for various design parameters such as what long-term material strengths and factors of safety to use. For the most part, these recommendations are based on the approaches used in AWWA and ASTM standards. Because they are heavily debated by subject-matter experts before their adoption, AWWA and ASTM standards carry considerable weight. However, the opinions of the report’s authors also are part of the study recommendations, to provide starting points for the needed debates. An example is a testing protocol suggested for determining whether a lining resists tearing. It is hoped that future AWWA standards will take into consideration the ideas presented in this study. The Pipeline Rehabilitations Standards Committee of AWWA is currently working on clarifying many of these issues.

APPLYING THE ASSESS-AND-FIX APPROACH

Two examples of applications of the assess-and-fix method are discussed in the report. The first involved the NDE inspection several years ago of 9 miles of corroding ductile iron pipe. In this example, a risk assessment approach was coupled with deterministic and statistical analyses to evaluate the likelihood of various failure modes for each stick of pipe in the 9-mile pipeline. Good pipe was differentiated from not-so-good pipe. Had the owner desired it, different rehabilitation methods could have been confidently used for different pipe reaches, with the expectation that many decades of additional service would have been achieved. Equally important, more than half the pipe was found to be in good condition and could have been left alone for another generation.

The second example involved an assess-and-fix demonstration in the City of Phoenix. This demonstration was limited in scope, involving approximately 500 feet of main, and was performed solely to demonstrate the method. The demonstration illustrated how easily a NDE tool can be pulled through a water main, once the main has been prepared for lining. The demonstration also illustrated the potential benefits of using NDE when performing water main rehabilitation. While the pipe was not badly corroded, multiple through-wall pits were detected by the NDE scanning. These through-holes justified a semi-structural lining rather than the non-structural lining which was specified for this main. Fewer future leak repairs would be expected had the more robust lining been applied, and the added cost might have been marginal.

FINDINGS AND RECOMMENDATIONS

The assess-and-fix method can be used today. The necessary technologies exist. Assess-and-fix is already offered in the marketplace, and has been demonstrated through this WRF study. All that is required are utilities that wish to employ it.

While there are technical issues to be figured out, they involve refinements rather than proofs of concept. By joining water main rehabilitation and NDE technologies together, both will advance: a better-defined rehab product is achieved and the NDE is completed cost-effectively. By targeting the bad portions of pipe, making use of the good portions of pipe, and spurring broader use of trenchless rehabilitation, the added cost of employing NDE should be recovered through lower infrastructure renewal costs, once the method becomes routine.

There is one missing ingredient in assess-and-fix implementation: one or more large utilities are needed that see the value in this method, adopt it, and push its development. By adopting this approach as part of a substantial capital improvement program, the assess-and-fix system of project delivery can quickly advance. Any large water utility should be capable of filling in the technical gaps, including:

- Standards, criteria, and test methods for linings
- Inspection and analysis methods for timely and more useful NDE assessments

Time to Think Outside the Trench

For run-of-the-mill water infrastructure renewal, there is arguably little that is accomplished through open-trench construction that cannot be accomplished with rehabilitation and other trenchless methods, but in the water industry, the adoption of trenchless has lagged. Uncertainty about the integrity of the old main and uncertainty about the value of the rehabilitated product are two of the reasons. An assess-and-fix approach helps remove these uncertainties.

If water engineers followed the example set by the wastewater community by defaulting to low-dig approaches, greater industry-wide adoption of trenchless would drive innovation, providing savings of money, time, and community impacts. The assess-and-fix marriage of rehabilitation and NDE assessment is a model for how this can be accomplished.

SUMMARY OF STUDY CONCLUSIONS

1. Condition assessment and pipe rehabilitation are both routinely used in the wastewater industry, where these methods are economically performed, well understood, and addressed by widely-accepted standards. Because water mains are more complex than gravity sewer mains, use of condition assessment and rehab in the water industry has lagged. By employing condition assessment as part of rehabilitation, an owner is able to select a lining method with confidence. Broad use of this method would lead to development of applicable industry standards and substantial economies of scale.
2. Remote-field testing (RFT) is currently the preferred technology for asset-and-fix application. High-resolution, accurate results have been validated by several independent tests, and the technique has been used on water mains for nearly 20 years. The tools and services currently available reflect this long experience. RFT tools are available for pipes ranging from 4-inch to 36-inch. Magnetic flux leakage (MFL) also provides meaningful data needed for assess-and-fix applications, but only recently has this method been applied to water main assessment.
3. A visual inspection should accompany most NDE assessments, as an aid to interpreting data. It is generally important to distinguish internal from external corrosion pitting, since internal corrosion will be largely stopped with the application of the lining, but external corrosion will continue. Video inspections using closed-circuit cameras are often performed prior to lining anyway, so this is not necessarily an added step or extra cost.
4. No method is perfect and no inspection is 100 percent, but most water distribution mains are “low-consequence” assets which don’t require perfect, precise analysis. With a combination of graphical data display and interpretation by a trained technician, adequate information should be available for routine assess-and-fix rehabilitation decisions within a reasonable time frame.
5. Because corrosion of iron pipe is a generally decelerating process, the future condition of a 50-year old pipe can be confidently forecast if its current condition is known. A pit that reaches 8 mm penetration after 75 years should grow by only 1 mm in the next 50 years. The fuzzy-logic pit growth model of Rajani, et al., (2011) can be used to predict the depth of future pits. This prediction model relies on information regarding historical pit growth for the pipe being assessed. If maximum pit size and pipe age are known, future pits sizes can be estimated. Information about the corrosivity of the environment is not required.

6. Pipe corrosion (both pitting and general) contributes to various types of failures, but is not the only influence or aging factor. Other factors to be considered in assessing failure risks include:

    - Wall thickness and pipe diameter
    - System pressures, pressure cycles, and surges
    - Potential for ground movement
    - Type of joint material
    - Material ductility

7. Three methods are provided for interpreting NDE data and evaluating the risk of main break:

    - **Statistic analysis** is useful for assessing the likelihood of different types of breaks and their association with various factors. As a data base of NDE data is built up, NDE data can also be used in these assessments.
    - **Deterministic analysis** can be used to forecast future pit size and calculate stress levels and residual safety factors. These analyses can be difficult to perform due to complex patterns of corrosion pitting, uncertainties about material strengths, and unknown strains created by pipe bending.
    - **Risk assessment** is a practical way of prioritizing and categorizing pipes based on their assessed condition, while also taking into consideration pressure, soil stability and other factors that contribute to breaks. While this method is somewhat subjective, its accuracy can be improved by comparing results to the statistical and deterministic methods.

8. The selected rehabilitation method should reflect the type of pipe break considered most problematic:

    - Class[2] IV (fully structural) methods are needed for pipes with insufficient remaining hoop strength
    - Class III (semi-structural), *tear-resistant* methods are appropriate if circumferential (beam) breaks are likely
    - Class II and III (semi-structural) methods are useful for stopping rust-hole leaks and joint leaks
    - Class I (non-structural) methods are appropriate if little external corrosion has occurred and the pipe has sufficient residual strength

9. In many cases, the tear resistance and water-tightness of lining products need to be tested in order to confirm that they meet the desired performance criteria. Samples for testing should be taken from mains lined in place, and tests should be performed under a pressure that reflects expected system conditions.

10. Several existing standards provide guidance for evaluating deteriorated mains and designing appropriate lining systems:

    - ASME B31G provides guidance for how closely-spaced corrosion pits may be analyzed

[2] Class I, II, III, and IV refer to the lining classifications of the AWWA M28 Manual of Practice.

- ASTM F1216 provides a formula for determining the maximum size of hole for which a Class II or III lining is appropriate. This standard also provides a formula for determining the hole that may be spanned by a lining.
- Various ASTM and AWWA standards provide guidance for determining the long-term material properties of plastic lining materials

The ASME and ASTM standards should be used somewhat cautiously, as they were not developed with water main lining in mind. Also, FEA performed by Brown, et al., (2014) found that ASTM F1216 was not always conservative.

11. AWWA currently has standards for two Class I systems (cement mortar and 1mm epoxy). Standards are needed for the other lining systems as well as guidance in evaluating the condition of mains from NDE data. Starting points for these standards are suggested in this study. Because small-diameter water mains are generally low-consequence assets, modest safety factors are suggested, particularly where a ductile system is provided.

12. A demonstration in Phoenix showed the practicality of performing NDE in middle of an lining project. Had the project been a true assess-and-fix project, a Class II lining would have been recommended rather than the Class I lining that was applied. This could have been easily accommodated with a spray-applied polyurea lining.

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Asset Management: Performance, Sustainability, and Resiliency Model Development

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Abstract

In the past five years, EPA and other agencies have been talking about sustainability and resiliency. Each of these areas has received attention on an individual basis. There are parameters and interdependencies which cross all three areas; performance, sustainability and resiliency. This paper will present the research to date on development of the governing parameters associated with each of the three areas. The basic asset management plan shall be viewed under the following distinct categories, performance management, sustainability management, and resiliency management. Parameters for performance will be selected which are the major contributors, based on: Industry Standards, Standard Practice, Research, Wide Use, Data Reliability, and Data Sustainability. Based on the evaluation matrix a performance index will be assigned. Each area will receive an indicator index of 1 (one) to 5 (five). The results of the research will be presented at the conference.

INTRODUCTION

Asset Management has become a major driver in the water industry for determining infrastructure management. The deterioration of the infrastructure has resulted in severe budget ramifications and a great impact on level of service for the water, wastewater and stormwater utilities. There has been advancement in asset management throughout the world during the past 15 years. In most cases the asset management models have dealt with condition and financial issues. In the US, there has been a great deal of focus on condition assessment, deterioration curves and triple bottom line analysis.

In the past five years, EPA and other agencies have been talking about sustainability and resiliency. Each of these areas has received attention on an individual basis. There are parameters and interdependencies which cross all three areas; performance, sustainability and resiliency.

This paper will present the research on development of the governing parameters associated with each of the three areas. The basic asset management plan shall be viewed under the following distinct categories, performance management, sustainability management, and resiliency management. The initial three areas will be defined as follows:

Performance	**Sustainability**	**Resiliency**
Structural Condition	Social	Assessment Plan
Internal Environment	Environmental	Risk Mitigation
External Environment	Economic	Recovery

Each area will receive an indicator index of 1 (one) to 5 (five).

Performance – Parameters will be selected which are the major contributors, based on: Industry Standards, Standard Practice, Research, Wide Use, Data Reliability, and Data Sustainability. Based on the evaluation matrix a performance index will be assigned.

Sustainability – Parameters will be selected which are the major contributors, based on the same measures listed above. Based on the evaluation matrix a sustainability index will be assigned. The EPA Guidance Document on Sustainability will be a guide in development of the index so that important regulatory support will be attained for the sustainability index.

Resiliency – Parameters will be selected which are major contributors, based on the same measures listed above. Based on the evaluation matrix a resiliency index will be assigned. Several existing tools will be reviewed and used appropriately in the matrix evaluation to develop the index.

Development of an overall three dimensional measure of service level index from 1 (one) to 5 (five), will take place which shows the relative impact of performance, sustainability, and resiliency.

Water utility asset management programs have been developed following the U.S. EPA and WERF core definition of maintaining a level of service at the lowest life-cycle cost and at an acceptable risk. Most utilities, however, only incorporate performance measures into their asset management plans. A holistic approach to asset management is more beneficial because it takes into account the short and long term goals of the utility and can provide better service socially, economically, and environmentally. To address this approach, the past focus on performance must be separated into performance, sustainability and resiliency. By separating the Asset Management Plan (AMP) into the three areas an index can be developed for each of the areas. A common definition must be developed for each of the areas to focus on the areas, as there are many conceptions of what performance, sustainability and resiliency mean. This project will build on past work at Virginia Tech and previous students Masters and PhD work.

In coordination with Virginia Tech's Sustainable Water Infrastructure Management (SWIM) laboratory and information from many utilities and subject matter experts, information and data is gathered which will result in development of parameters and index's for performance, sustainability and resiliency. A review of all existing performance, sustainability and resiliency academic work and industry models has been studied and evaluated. The goal is to identify all parameters required to determine an index on a scale of 1 – 5, for a utility's performance, sustainability and resiliency. Then the weighting for each index will be determined to indicate an overall index for the utility by combining all 3 indices.

BACKGROUND

Over the past decade, many utilities, organizations, and regulators in the United States and world- wide have developed and published resources relevant to infrastructure asset management for water, wastewater and stormwater. EPA, AWWA, WEF, ASCE and others in the United States have urged utilities to move from a reactive role in asset management to a proactive role. Asset management is sometimes defined differently by these entities and there are only some aspects of AMP's which are common among them. Each has addressed performance, sustainability and resiliency, but in different ways, and based on a specific definition, which in some instances is not focused on water, wastewater, and stormwater conveyance systems.

Holistic Asset Management Framework

The asset management framework developed by Dr. Sinha incorporates basic elements that build on and complement one another to provide sustainable municipal infrastructure asset management. Unlike other asset management structures, this framework links standard asset management concepts, information systems, and sustainable and resilience management practices. Ideally, this framework provides utilities with a support system that handles short and long term holistic asset management planning (Sinha & Eslambolchi, 2006). The framework is outlined in **Figure 1**.

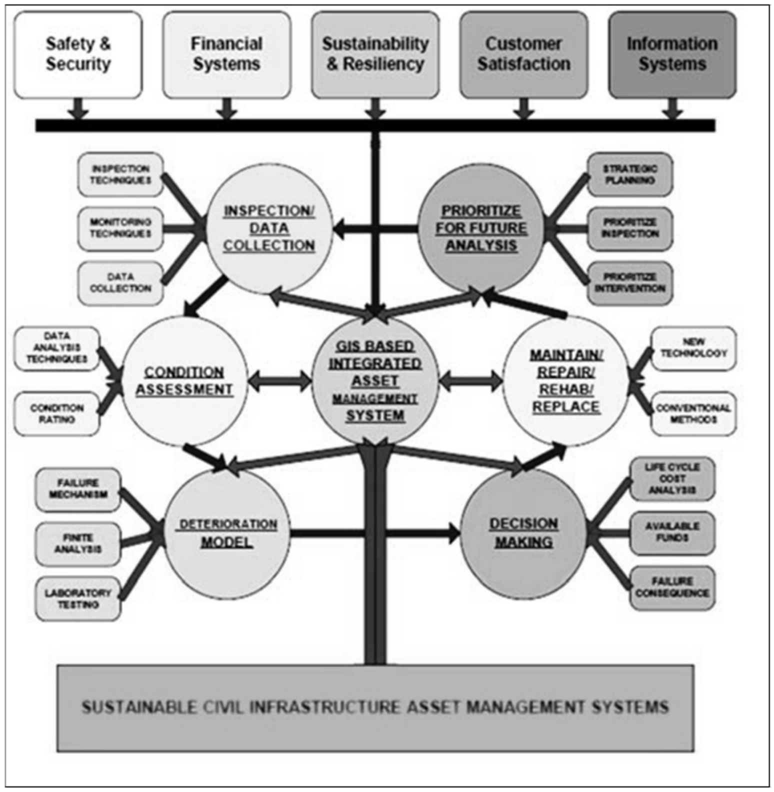

Figure 1: Holistic asset management framework. (Gay & Sinha, 2013

The holistic asset management framework incorporates various aspects of asset management such as data collection, condition assessments, decision-making, repairs and maintenance, and future priorities into its strategic model. Like many asset management frameworks, most of the components in the framework support performance management. Alternatively, the holistic asset management framework incorporates sustainability and resilience management concepts into the asset management discussion (Gay & Sinha, 2013). Sustainability and resilience management, explained more in the next section, are needed for an asset management framework to be truly holistic. Performance focuses on the physical functioning of assets as they pertain to providing the desired LOS. Sustainability and resilience management concepts bring goals and standards to a utility that include the well being of the community and the environment as well as preparation for disaster.

Performance Management

Performance Management is defined as managing the infrastructure to minimize the total cost of owning and operating the system, while delivering acceptable service levels. Performance of a utility can be defined in terms of service life and reliability. Both of these criteria rely on factors involved with the structural condition, internal environment and external environment. Parameters must be identified and data collected from utilities for each parameter.

The following list of parameters has been identified for water systems for performance are shown in **Table 1.**

Table 1.Performance Parameters Water Systems

	Parameters	Attributes
External Environment	Coating	yes/no
	Cathodic Protection	yes/no
	Soil Type	A-1; A-2; A-3; A-4-A-5; A-6-A-7
	Moisture	Poor Drainage; Fair Drainage; Good Drainage
	Stray Current	Yes/No
	Flooding	0-1 per year; 1-5 per year; 5-20 per year; 20-100 per year; >100 per year
	Dynamic Load	Unpaved; Non-National Highway System; National Highway System; Interstate; Railroad/Airport
	ADT	0-10; 10-100; 100-500; 500-2000; >2000
	Disturbances	Yes/No
Internal Environment	Coating	yes/no
	Water pH (Baylis Curve)	5-6; 6-7;7-8; 8-9; 9-10
	Design Operating Pressure	50-100 psi; 100-200 psi; 200-300 psi; 300-400 psi; 300-400 psi
	Pressure Surge frequency	0-1 per year; 1-5 per year; 5-20 per year; 20-100 per year; >100 per year
Structural Condition	Lining	Yes/No
	Lining Type	Non-structural; Semi-Structural; Structural
	Tuberculation	yes/no
	Dissimilar Materials	yes/no
	Surge Control Valves	Yes/no
	Pump Control Valves	No Valve; Suring Check Valve; Other
	Age	
	Material type	
	Joint type	

A questionnaire was sent to many utilities, both public and private to collect the data for each of the parameters. The results showed what data is available and what data is not presently collected by most water utilities. Based on the statistical representation of the data, a determination was made of the applicable parameters to be included in the development of a performance index dealing with potable water. These results will be completed and presented in the presentation at the 2015 Pipelines conference.

The following list of parameters has been identified for wastewater systems for performance shown in **Table 2**.

Table 2.Performance Parameters Wastewater Systems

	Parameter	Unit	Range
External Environment	Ground Cover	Type	unpaved road, gravel, grass, dirt, loose particle material;
	Groundwater Table	level	Below pipe; close to pipe <2ft; Slightly above 2-5ft; above pipe>4ft;
	Location (Traffic)	level	light >50ft from road or railway; medium 50ft from road or railway; heavy 20ft from major road or railway;
	Pipe Depth	ft	>18; <4
	Pipe Slope	%	0-5
	Soil Corrosivity	level	
	Soil Type	Type	gravel, coarse sand, fine sand, silt, clay
	Tidal Influences	Yes/no	Yes; No
Internal Environment	D/d (flow depth over diameter)	%	0-100
	Flow Velocity	f/s	0-5
	Pipe Surcharging	Level	Frequent; Occasional
	Wastewater pH	pH	0-9
Structural Condition	Density of Connections	level	Very Dense (>5 per 100ft), Dense (4-5 per 100ft)
	Maintenance Frequency	Level	Very Often, cleaning and Inspecting every 1-3 years; Regularly, 3-5 years; Rarely, >5 ; Never
	Pipe Age	Year	0-130
	Pipe Condition	Level	0-5(also comment the supporting criteria)
	Pipe Diameter	inch	0-60
	Pipe Length	ft	0; 500 ft

A questionnaire was sent to many utilities, both public and private to collect the data for each of the parameters. The results showed what data is available and what data is not presently collected by most water utilities. Based on the statistical representation of the data, a determination was made of the applicable parameters to be included in the development of a performance index dealing with wastewater. These results will be completed and presented in the presentation at the 2015 Pipelines conference.

Sustainability Management

Sustainability management is maintaining a system that continuously satisfies need without compromising the ability of future generations to satisfy their own needs from the Triple Bottom Line (TBL) perspective (Gay & Sinha, 2013). Utilities in other countries, like the UK and Australia, have begun to adopt sustainable practices into their AMPs (Marlow, Beale, & Burn, 2010 and Rees, Young, & Richardson, 2009). It has become globally important to address new world challenges linked to climate change, population growth, damage to ecosystems, and reduction of greenhouse gases. Because AMPs are always evolving, adding sustainability management to AMP goals and objectives can help utilities meet these challenges. Sustainability management can be implemented in small steps (Marlow, 2010). The most important overarching goal of sustainability management is that in each decision-making step a TBL perspective be considered (Kenway, Howe, & Maheepala, 2007). Though the UK and Australia may be global leaders in sustainable asset management planning, the US EPA has designed a handbook for utilities to start to imbed sustainable goals into their planning and management of their water and wastewater infrastructure. They believe that the core mission of water and wastewater utilities is to provide clean and safe service that includes not just public health but environmental health and economic sustainability (EPA, 2012). The handbook helps utilities create goals and implement practices that incorporate TBL thinking into organizational practices.

Many researchers agree that setting TBL objectives within an AMP into can create more sustainable services. However, because asset management helps in long term planning, planning for future wastewater needs and addressing future problems becomes part of everyday thinking. Nevertheless, adding TBL principles to asset management help utilities address emerging issues due to climate change along with changing populations (Marlow, et al, 2010).

Sustainability Management considers the ability of the Infrastructure system to operate at pre-defined levels of service for indefinite time. The 3 areas of consideration in sustainability management are Social, Economic, and Environmental. Parameters must be identified and data collected from utilities for each parameter.

The following list of parameters has been identified for water systems for sustainability shown in **Table 3**.

Table 3.Sustainability Parameters Water Systems

	Parameters	Unit	Comment
Economical	System energy use	BTU	Total Energy used
	Asset Management Plan	Yes/No	if yes, please provide details (condition assessment practices; models, tools used)
	Utility Revenue	$	
	Utility Expenditure	$	
	Revenue saved for future renewal	$	
	Revenue spend for capital improvement	$	
	Revenue spend for operation	$	Includes renewal activities, routine maintenance, equipments, salaries, etc.
Environmental	Source of Energy	Type	Type of Energy source (gas, oil, solar, etc.)
	Source of Water	Type	
	Source Water Capacity	Gallons	Water drawn for treatment
	Water Loss	Gallons or %	unacounted water loss
	Pipe Breakages	Number	
Social	Served Water Capacity	Gallons	Water supplied to customer
	Meeting Demand	%	Expected increase in water supply
	Service provision	%	Expected increase in customers in 10 years
	Customer complaints		
	Customer Education Outreach	Yes/No	Being Practiced? (Y/N)

A questionnaire was sent to many utilities, both public and private to collect the data for each of the parameters. The results showed what data is available and what data is not presently collected by most water utilities. Based on the statistical representation of the data, a determination was made of the applicable parameters to be included in the development of a performance index dealing with potable water. These results will be completed and presented in the presentation at the 2015 Pipelines conference.

The following list of parameters has been identified for wastewater systems for sustainability, shown in **Table 4.**

Table 4.Sustainability Parameters Wastewater Systems

	Parameters	Unit	Comments
Economical	System energy use	BTU	Total Energy used
	Asset Management Plan	Yes/No	if yes, please provide details (condition assessment practices; models, tools used)
	Utility Revenue	$	
	Utility Expenditure	$	
	Revenue saved for future renewal	$	
	Revenue spend for capital improvement	$	
	Revenue spend for operation	$	Includes renewal activities, routine maintenance, equipments, salaries, etc.
	Sewer Flow	gallons	
	% sewer capacity		
	I & I	gallons	
Environmental	Source of Energy		Type of Energy source (gas, oil, solar, etc.)
	Sewer Overflows	#	
	Sewer Backups	#	
Social	Customer complaints	#	
	Customer Education Outreach	Y/N	Being Practiced? (Y/N)
	Service provision	#	Expected increase in customers in 10 years

A questionnaire was sent to many utilities, both public and private to collect the data for each of the parameters. The results showed what data is available and what data is not presently collected by most water utilities. Based on the statistical representation of the data, a determination was made of the applicable parameters to be included in the development of a performance index dealing with wastewater. These results will be completed and presented in the presentation at the 2015 Pipelines conference.

Resilience Management

Resilience management is the ability to avoid, reduce, mitigate, and ultimately recover from the effects of natural, accidental, or malevolent incidents with minimal impact on end-users (Gay & Sinha, 2013). Resilience management is often the most difficult management structure to add to any utility AMP. It begins with identifying what hazards to which the utility could be exposed, and then making specific goals to address them. A deterioration model in the holistic asset management framework plays a major role in resilience management (Gay & Sinha, 2013). Deteriorating assets are more susceptible to disastrous events. Resilience management has 3 components: assessment plan, risk mitigation, and recovery.

The following list of parameters has been identified for resilience, shown in **Table 5.** These parameters apply to both water and wastewater systems.

Table 5.Resiliency Parameters Water & Wastewater Systems

	Parameters	Units
Assessment Plan	Pipe Location	
	Proximity to other Assets (including pipes)	feet
	Proximity to Hazards	feet
	Material	
	Diameter	inches
	Age	
	No. of nodes per demand point	
	Likeliehood of Failure	
	Quality of other Utility Record	
	Resource availability material, personnel, equipment	Y/N
	Preventive Measures Plan	Y/N
	Emergecy Preparedness Training	Y/N
	Financial Impact to Private Property(in case of disruption)	$
Recovery	pipe network redundancy	Y/N
	Coordination Plan with other agencies	Y/N
	Acceptable recovery cost to original LOS	$
	Acceptable recovery time to original LOS	$
	Access to pipe	Y/N
	Budget at dispense for emergency per area or in total(in $)	$
Risk Mitigation	Condition	
	Hazard Type (earthquake, theft)	
	Type of nearby property	
	Post Event Plan Available	Y/N

CONCLUSION

Previous work at Virginia Tech by Masters and PhD students has developed models dealing with performance of water and wastewater assets. Data from those models has been instrumental in the development of the parameter questionnaires for performance parameters. Those models were run with data from utilities and produced performance measures for the data collected. Parameters for the questionnaire for Performance were selected as major contributors, based on Industry Standards, Standard Practice, Research, Wide Use, Data Reliability, and Data Sustainability. With Sustainability, the EPA Guidance Document on Sustainability was a guide to develop the questionnaire parameters. There are several existing Resiliency references that were used to develop the parameters used on the questionnaire for Resiliency. The information collected from the questionnaires will result in identification of the parameters most relevant to determining an index for performance, sustainability, and resiliency.

A model will be developed to give an index from 1 – 5 for each of the areas: Performance, Sustainability, and Resiliency. Several utilities which have well developed asset plans will be polled to review and give verification of the practical use of the index system rating for their system. The results of the index model will be presented at the 2015 Pipelines Conference. The relationship between deterioration curves and performance, sustainability, and resiliency are shown in **Figure 2.**

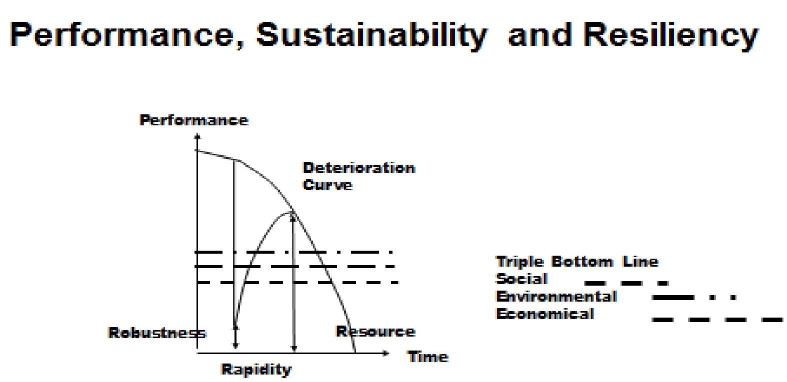

Figure 2.Curves of Deterioration Performance, Sustainability, Resiliency

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Finite Element Modeling of Full-Scale Concrete Manholes under Soil Pressure

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Abstract

Large amount of manholes in the United States are suffering from serious structural decay and are in need of immediate rehabilitation. Among various candidate techniques, spray-on-place lining is one of the promising techniques for manhole rehabilitation. Based on a new manhole rehabilitation classification system proposed by the authors, epoxy liner is considered as semi-structural material and relies on residual strength of existing manhole structure to withstand external loads. In this study, a two dimensional axisymmetric finite element model for concrete manholes is developed using ABAQUS and calibrated to an existing full-scale manhole experiment. The calibrated FEM manhole model is then used to model the structural behavior of the manhole with epoxy liner under in-situ soil pressure. Results show that concrete residue strength is needed for deteriorated manhole repaired with epoxy liners to resist in-situ soil pressure load.

INTRODUCTION

Manholes are access points to underground infrastructures. Large amount manholes are deteriorated due to several reasons, such as design and construction defaults, climatic or chemical agent damage, corrosion, accidental overloading or impact, fires, or earthquakes (Nour et al. 2007). The United States Environmental Protection Agency (USEPA) estimated that 3.5 million manholes are suffering from serious structural decay and are in need of immediate rehabilitation or replacement (Sever et al. 2013). Considering the potentially multi-billion dollar market, it is not surprising that there are already numerous materials and methods available for manhole rehabilitation. Concrete patching, using manhole linings, corrosion protection and replacing the whole or some parts of the manhole are some of the methods used to solve manhole problems. Among all the available methods, No-dig manhole rehabilitation attracts great interest due to its low cost and minimum interference on traffic. In this method instead of replacing the manhole structure, different materials are used to retrieve manholes' capability to withstand the applied loads and infiltration.

Several experimental studies have been conducted to evaluate different properties of manhole rehabilitation materials (Render et al. 2004; Ahn et al. 2009; IKT 2012). But to the authors' knowledge, numerical modeling of manhole structures is limited. Sabouni conducted her doctoral research on load and deformation of new precast concrete manholes in 2008 (Sabouni 2008). In her study, three manholes including one reinforced and two unreinforced concrete manholes were fully instrumented with strain gages and soil pressure cells and tested in a soil test pit under different load configurations specified in the standards. The manholes were simulated using PLAXIS to analyze the soil pressure, bending moment, and strains (Sabouni and El Naggar 2011). A field demonstration project was performed in Cleveland, OH in a project funded by EPA to evaluate the performance of cured-in-place pipe lining rehabilitation method (Matthews 2012). According to the report the project resulted in successful demonstration of an innovative Class IV (fully structural) water main rehabilitation technology.

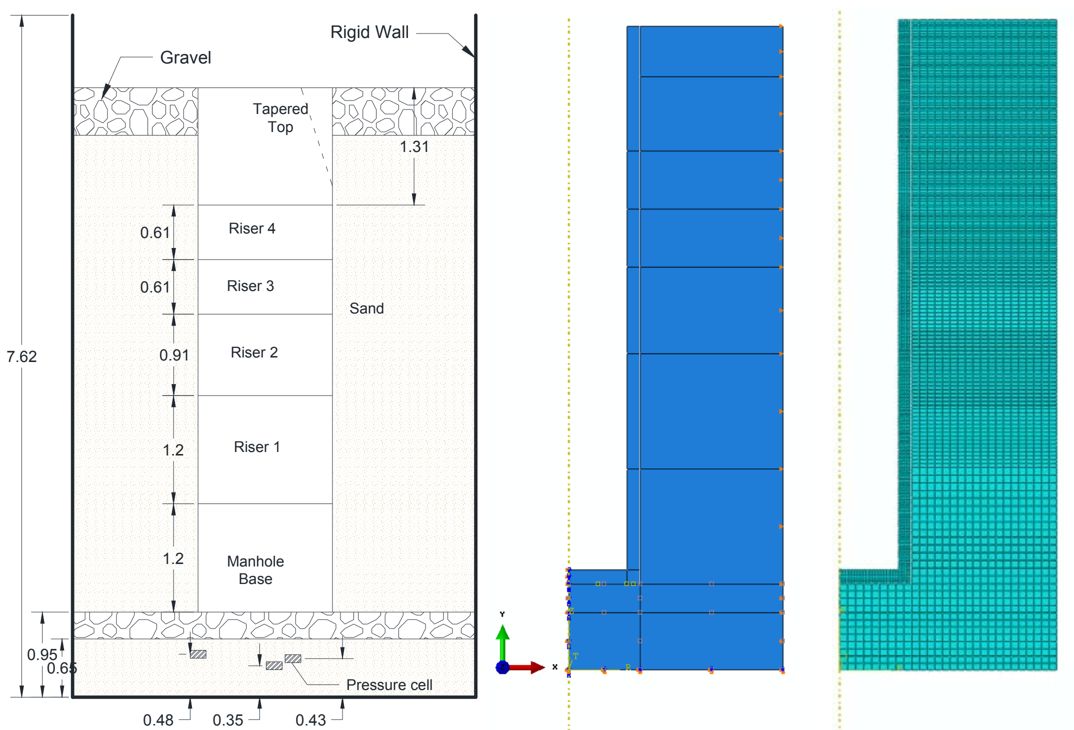
Behavior of epoxy coated concrete was studied by testing coated concrete specimens in flexural and compression tests and by finite element simulation (Riahi et al. 2014). The knowledge of previous study and existing experimental data on manholes is adopted to create a finite element model of concrete manhole with epoxy liner. The model of the manhole structure and surrounding soil is generated in ABAQUS and then validated with analytical calculations and the results from an existing full scale manhole experiment. The calibrated model is then used to study the behavior of the epoxy liner under lateral pressure from the surrounding soil.

FINITE ELEMENT MODEL

ABAQUS/standard is used to model a full scale manhole. A typical 1.2m diameter concrete manhole structure tested in a previous laboratory study (Sabouni and El Naggar 2011) was selected for the manhole simulation. This manhole was fully instrumented with strain gages and pressure cells and tested under various load configurations in a large laboratory test pit. The available test data allows for verification of the developed ABAQUS manhole model. After the new concrete manhole model was validated, deteriorated manhole with epoxy liner was then modeled to study the structural capacity of epoxy liner for manhole rehabilitation.

The concrete manhole consists of one 1.2m high monobase, four risers, and one tapered top as shown in Figure 1a. The wall thickness of the monobase is 139mm with a 1.219m inner diameter and a 150mm thick nonreinforced base. Height of each manhole part is illustrated in Figure 1a. The total height of the manhole is 5.89m. The manhole was tested in a soil pit with rigid walls and floor. The soil test pit had a base of 4.5m by 4.5m and height of 7.26m. A layer of 0.65m sand was located at the bottom with a layer of 0.3m gravel on top of that. The manhole base was placed on the gravel layer and the manhole walls were surrounded by 5.33m concrete sand. The top layer was 0.53m thick gravel (Sabouni 2008). The manhole was installed following the standard installation procedure specified by the Ontario provincial standard specification (OPSS516) (MTO and MEA 2005)

The installation simulation was performed by following all the procedures in the experiment. A 2D axisymmetric model was generated in ABAQUS. CAX4R element was used to model both the concrete and the soil. In the initial step the layer of sand and gravel beneath the manhole were generated and geostatic stress was obtained for those two layers. Stage construction was started by adding the monobase and sand layer with 1.2m height at the first stage. Each stage was defined in the model as one step in which the soil layer and the manhole riser were activated. The interaction between the soil and concrete was modeled using frictional model with normal and tangential behavior. The friction formulation used for tangential behavior was "Penalty" with friction coefficient of 0.5. The normal behavior was modeled with "Hard" contact pressure over closure approach, without allowing separation after contact. The tapered part of the manhole was ignored since it did not affect the results significantly (Sabouni and El Naggar 2011). The geometry and mesh used for the model are shown in Figure 1.



a. physical manhole test (unit m) b. ABAQUS model c. FEM mesh

Figure 1: Physical full scale manhole laboratory test and ABAQUS model

The boundary conditions were applied as following. The soil was restricted in horizontal direction at the right most vertical boundary. Fixed condition was applied to the bottom soil boundary. Axisymmetric boundary condition was applied to the concrete and soil along the symmetry axis. The concrete and soil interface was modeled used the interaction described in the above section. The material properties used for the simulation of the soil layers are presented in table 1. The data was obtained from the experimental program (Sabouni and El Naggar 2011).

Table 1: Soil properties

Soil Type	Unit Weight (kN/m^3)	Modulus of Elasticity (kN/m^2)	Angle of internal Friction (°)
Sand	19	$5.2x10^5$	39
Gravel	23	$6.8x10^5$	42

Concrete damaged plasticity model (ABAQUS analysis user's manual), available in the ABAQUS material library, was used to simulate the manhole structure. According to the ABAQUS theory manual, concrete damaged plasticity constitutive theory aims to capture the effects of irreversible damage associated with the failure mechanisms that occur in concrete and other quasi-brittle materials. The input parameters for concrete, presented in Table 2, were determined from laboratory tests on bare concrete cylinders and bare concrete beams (Riahi et al. 2014). For parameters that cannot be directly determined from these tests, values were estimated from literature. For Epoxy lining a bilinear material model was selected with elasticity modulus of 5019 MPa, yield stress of 31 MPa and ultimate strength of 58 MPa. Material properties of the epoxy were obtained from manufacturer's data sheet.

Table 2. Concrete Material Property

Modulus of Elasticity (GPa)		36
Poisson Ratio		0.2
Density (kg/m^3)		2400
Plasticity	Dilation Angle (°)	38
	Eccentricity	0.1
	f_{b0}/f_{c0}	1.16
	K	0.667
	Viscosity Parameter (s)	10^{-7}

MODEL VALIDATION

Results obtained from the 2D axisymmetric simulation of the manhole in ABAQUS are validated using analytical solutions and the results from full scale manhole experiment. The comparisons of the results are as follows.

Soil Pressure Distribution Under Manhole Base

Pressure distribution directly under the manhole base is shown in Figure 2. As it is shown, the pressure under the center from ABAQUS model is 74.5 kPa, which is approximately equal to the pressure induced by the weight of the manhole, 74.79 kPa. By moving towards the edges of the manhole, the soil pressure increases. This can be explained by the interaction between the manhole wall and the surrounding soil. The friction between the soil and manhole structure causes some of the weight of the soil been carried by the manhole and increases the pressure at the edges. Comparison of the results with PLAXIS simulation by Sabouni and El Naggar (2011) shows that the total trend of pressure distribution under the manhole base is similar with a difference about 20 kPa at the manhole base center. This difference is caused by the different structural elements (shell element in Plaxis and solid element in ABAQUS) used in the simulations.

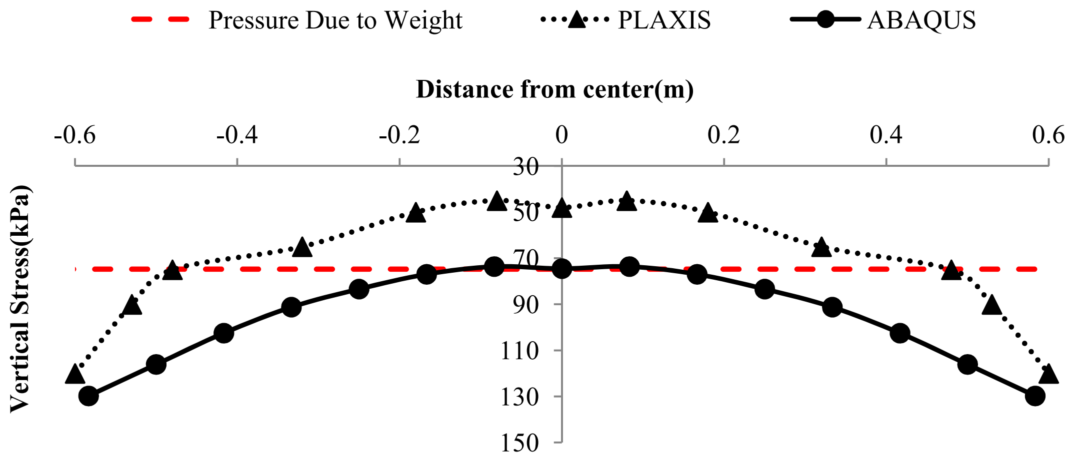


Figure 2. Pressure distribution under manhole base

Lateral Soil Pressure

Lateral soil pressure acting on manhole structure is shown in Figure 3 and the result is compared to active and at rest lateral earth pressure predicted by Rankine earth pressure theory. The results can also be validated by the results obtained from full scale manhole experiment. The pressure cell located at depth of 1.44m shows a pressure equal to 10.2 kPa which is close to the results obtained from ABAQUS simulation equal to 10.5 kPa.

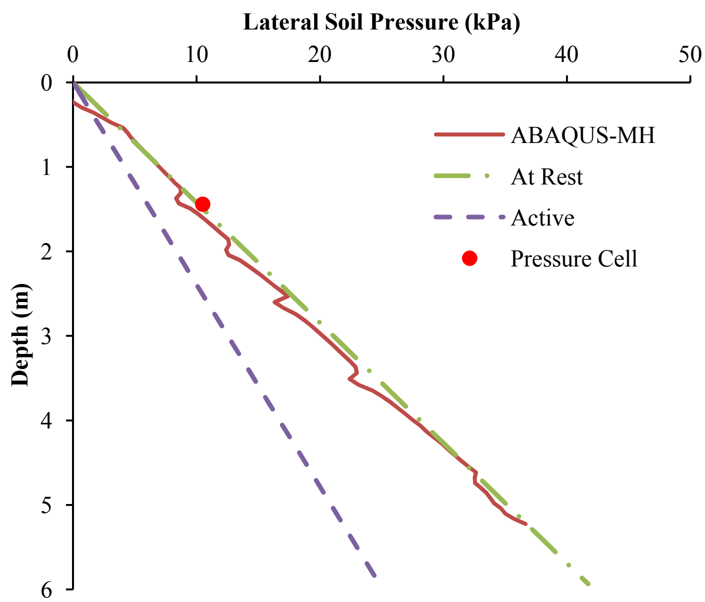


Figure 3. Lateral soil pressure in depth

Moment in manhole base

The moment in the manhole base can be calculated by using clamped plate theory (Reddy 1999). According to plate theory the radial and angular moment (M_r and M_θ)

in a circular clamped plate with radius "R" under uniform pressure of "p" at a distance of "r" is calculated as:

$$M_r = \frac{pR^2}{16}\left[(1+\upsilon) - (3+\nu)\frac{r^2}{R^2}\right] \quad (1)$$

$$M_\theta = \frac{pR^2}{16}\left[(1+\upsilon) - (1+3\nu)\frac{r^2}{R^2}\right] \quad (2)$$

The above equations indicate that the maximum moment occurs at the center of the disk and it is equal to:

$$M_{max} = \frac{pR^2}{16}(1+\upsilon) \quad (3)$$

Figure 4 compares bending moment in the manhole base calculated by clamped plate theory with the results obtained from ABAQUS. The pressure used for calculating the moment is the average pressure beneath the manhole base obtained from simulation. The maximum bending moment which is located at the center of the manhole base is in good agreement with the calculations from clamped plate theory. By getting closer to the edges the results from the simulation differ from the results from calculations which can be due to the fact that the manhole base is not acting like a clamped plate and some movements may occur at the edges. Comparison of the maximum moment obtained from PLAXIS simulation (Sabouni 2008) is also shown in Figure 4. There is a 32% difference between the results which can be due to difference in magnitude of the pressure under the manhole base from different simulations.

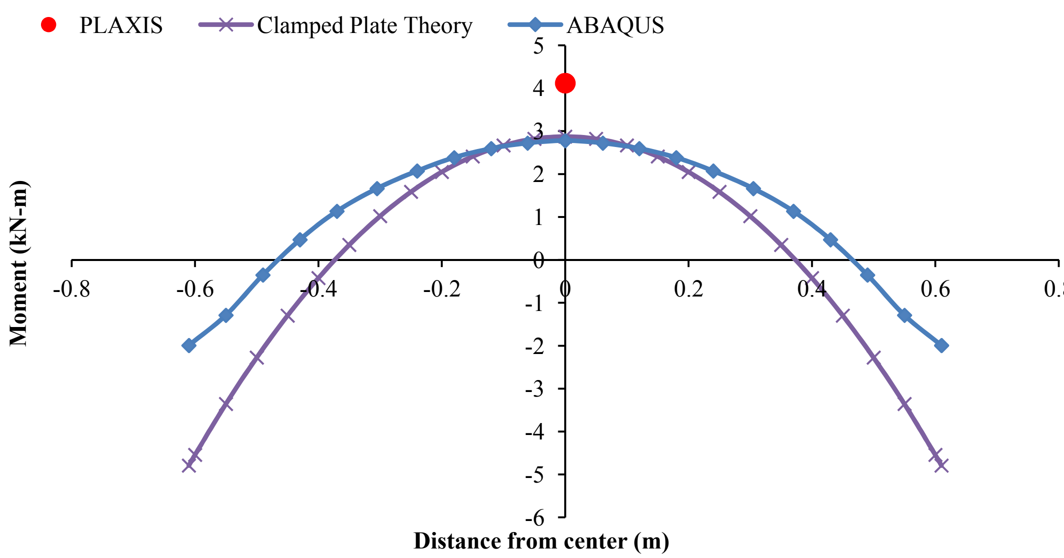


Figure 4. Moment in manhole base

Although perfect match is not obtained from the ABAQUS simulation and PLAXIS simulation of the existing full scale manhole experiment, regarding analytical calculations and acceptable match in total trend of the results, the generated model can be a good comparison base to study the effect of applying epoxy coating inside of the manhole structure.

FINITE ELEMENT MODELING OF EPOXY LINED MANHOLE

The above verified concrete manhole model was modified to simulate behavior of a deteriorated manhole with epoxy liner. Two manhole conditions are considered. In the first one it was assumed that the concrete structure was partially deteriorated and epoxy lining was applied inside of the manhole structure; in the second condition the worst case scenario was presented in which the whole manhole structure was deteriorated without any structural support and the loads are carried only by the epoxy liner.

The first case was simulated by reducing the Young's modulus of the concrete to one tenth of its original value and the second case was simulated by replacing the concrete material of the manhole with soil. In both cases a layer of 6 mm epoxy liner was applied to inside of the deteriorated manhole. The interaction between the coating and concrete was considered as frictional behavior with a friction coefficient of 0.5 in tangential behavior and hard contact in normal behavior.

Results And Discussion

By replacing the concrete in manhole structure with deteriorated concrete and adding epoxy liner, lateral deformation of the manhole increases as expected. Figure 5 compares lateral deformation of epoxy liner in two cases. Case I refers to the situation of the deteriorated concrete with one tenth of concrete stiffness and Case II refers to the situation of replacing the concrete structure with soil. It should be mentioned that in these simulations the only load acting on the manhole was lateral pressure of surrounding soil. The other loads such as water pressure and traffic are not considered in this study. Radial deformation of the manhole at different depths is also calculated by using theory of elasticity for pipe under uniform pressure (Saada 1993). According to the theory of elasticity, radial deformation at any distance of "r" from center in a hollow cylinder with inside diameter of "a" and outside diameter of "b" under peripheral pressure of "P" can be obtained using following equation:

$$u_r = \frac{1-\nu}{E}\frac{-b^2P}{b^2-a^2}r - \frac{1+\nu}{E}\frac{a^2b^2P}{b^2-a^2}\frac{1}{r} \quad (4)$$

The pressure used in the above equation is obtained from the ABAQUS outputs at the chosen depth. The results from analytical calculation are in good agreement with results obtained from ABAQUS simulation.

The lateral soil pressure profiles of the two cases are shown in Figure 6. It can be seen that soil pressure decreases by reducing elastic modulus of the concrete in the case of deteriorated manhole. However, the soil pressure is still above active pressure state and no plastic strain is observed in the model. The soil pressure profile of the completely deteriorated manhole moves beyond active state which indicates an active state soil failure. The soil failure can also be observed in plastic strain development in the soil which is not shown here.

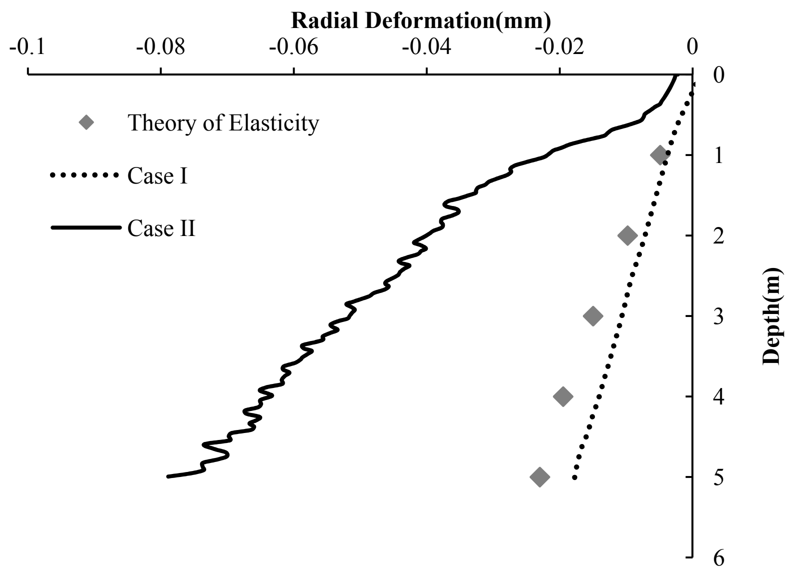


Figure 5. Deformation of the Epoxy coating

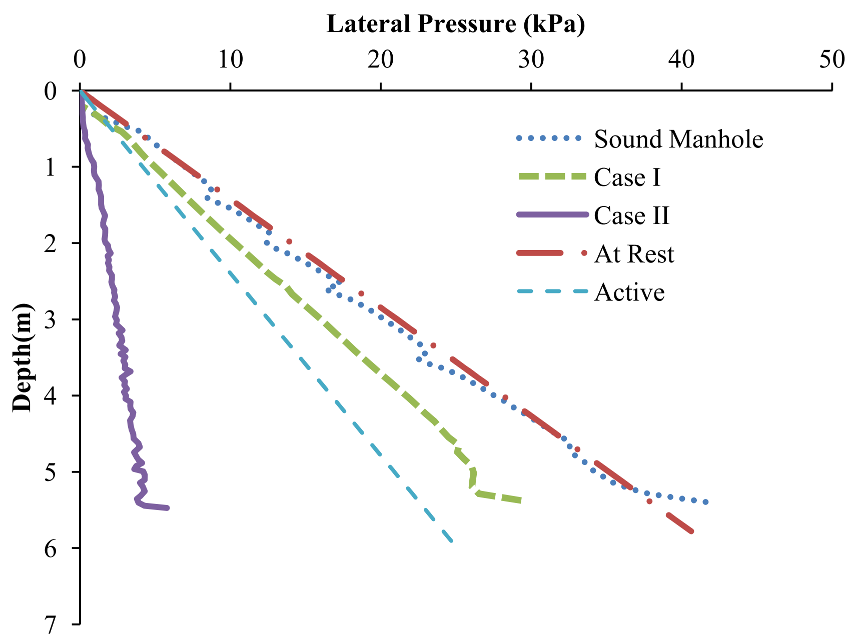


Figure 6. Lateral Pressure in depth

Deformation of the epoxy liner base in case II is shown in Figure 7. The maximum deformation is located at a distance of 0.52m from the center of the manhole. The maximum observed strain is 1.6×10^{-4} which is smaller than the plastic strain. Soil Pressure distribution under the epoxy liner base (Case II) for this case is also shown in Figure 8. The pressure beneath the manhole in this case is decreased significantly as compared to new concrete manhole. As the self-weight of the epoxy liner is small, the soil pressure is mainly caused by the load transfer from the soil due side friction.

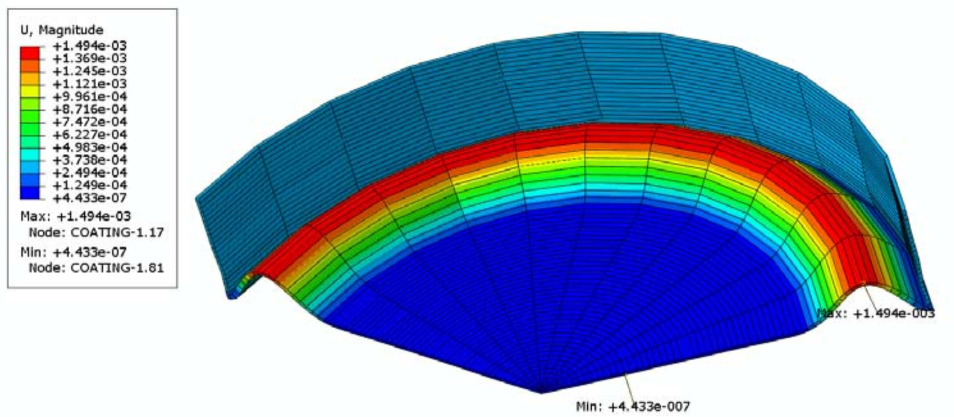


Figure 7. Deformation of the epoxy coating at the base of the manhole for Case II (m)

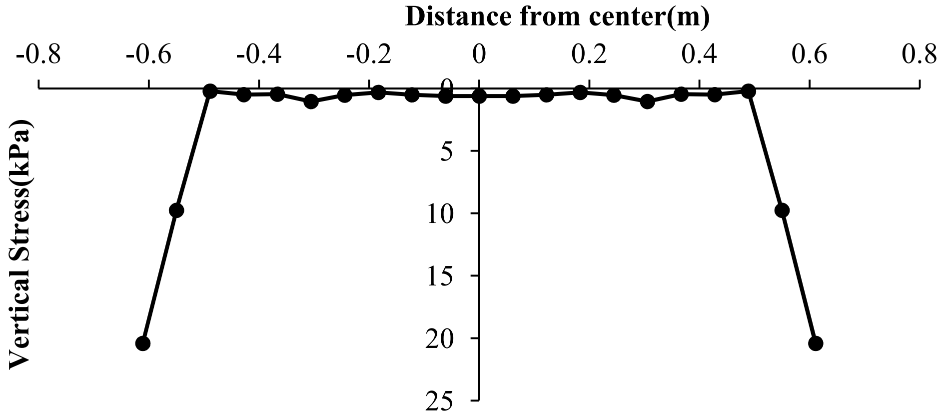


Figure 8. Pressure distribution under manhole base for Case II

CONCLUSION

A finite element model of manhole structure and surrounding soil was generated in ABAQUS and it was validated by the means of an existing full scale manhole structure and analytical calculations. The calibrated model was used to study the behavior of epoxy lining inside of the deteriorated manhole structure. The soil pressure along the manhole depth is close to at-rest soil pressure. The soil pressure beneath manhole is close to in-situ vertical stress in the soil. Partially deteriorated manhole with one tenth of remaining modulus can be repaired with the 6mm epoxy liner. The manhole with epoxy liner only fails due to active soil pressure failure. The developed model can be used to perform further analysis on behavior of the epoxy liner under different loading conditions and with different percentage of deterioration in manhole structure. More studies are needed for exact soil pressure distribution under concrete manhole base.

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Comparative Analysis of Geopolymer Technology for Sewer System Rehabilitation

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Abstract

Asset owners and engineers throughout the U.S. and the world are in search of cost-effective and environmentally friendly solutionsthat serve infrastructure issues. One of the most critical areas of concern is wastewater piping and structures. It is well known that corrugated metal pipes used in storm-water structures are corroding and microbial induced corrosion of sanitary sewers of various materials results in structural concern.Geopolymers have long been known to provide enhanced physical performance to traditional cementitious binders with the addedadvantages of significantly reduced greenhouse emissions and superior chemical resistance. Geopolymers are ceramic polymertechnology that creates a chemical material similar to natural stone that is superior to traditional Portland cement and shotcretematerials. However, they have not generally been contractor-friendly. This paper reviews a geopolymer mortar system that has been used in the U.S. since 2011 and is becoming a preferred solution fortrenchless rehabilitation. The system is spray cast either by rotary nozzle or via traditional shotcrete delivery systems inside ofexisting structures to create whole new structures which do not depend on the existing structure, just using it as formwork. This paperdiscusses competitive advantages over other trenchless repair solutions such as spiral wound, slip-lining and CIPP through specificcase studies including a corrugated metal storm drain rehabilitation in Hidalgo, Texas along with the repair of a stone sewer system inCincinnati, Ohio.

1.0 INTRODUCTION

As the state of infrastructure around the world decays, more cost effective solutions to repair large diameter pipe systems are required. Typical dig and replace technology is often not practical as in most urban areas these degrading pipes are located directly under other critical infrastructure such as major roadways, buildings, or other assets. As the diameter of these pipes become larger (>48 inches), the cost of many of the traditional trenchless technologies becomes exponentially more expensive and often requires significant excavation around access points that present additional issues related to community disturbance, traffic control, noise and general disruption. For example, if a 48-inch diameter sewer pipe were located in the center of town and a standard 30 or 36 inch manhole was the access point, in order to perform a CIPP (Cured-In-Place-Pipe) repair it would be necessary to excavate an access hole of at least the 48 inch diameter. While other techniques such as slip-lining would require even greater excavation for an access hole to install new liners. Additionally, with many of the standard so called trenchless repair technologies other issues related to either the shape (round, arched, elliptical) or the layout (straight, curved, bends of various radius) can make these repair technologies unpractical (Buczala,1990) (Osborn, 2010).

Over the last decade additional trenchless technologies have been developed to help fill the need for larger diameter pipe repairs at effective costs with little or no excavation requirements and minimal community disruption. One such technological advance is the use of centrifugally cast geopolymer mortars to create a new pipe inside the existing old pipe (Henning, 2012). This techniques allows for a cementitious pipe to be created within the existing structure, using the existing pipe as a form, and can be designed such that a new fully structural pipe is created. The flexibility of the technique allows for pipes of all shapes and layouts to be repaired either using automated mechanical casting or manually controlled material placement. The equipment necessary can easily fit down standard manholes and all excavation can be avoided if there are access points at least every 800 linear ft.

The benefit of geopolymer mortars as compared to traditional Portland cement (OPC) materials is detailed in the following discussion. Additionally, case studies are included.

2.0 GEOPOLYMERS

Geopolymer is a term originally coined by French researcher Joseph Davidovits to describe a class of "cement" formed from aluminosilicates. While traditional Portland cement relies on the hydration of calcium silicates, geopolymers form by the condensation of aluminosilicates. The kinetics and thermodynamics of geopolymer networks are driven by covalent bond formation between tetravalent silicon and trivalent aluminum. The molar ratio of these key components along with sodium, potassium, and calcium have been shown to affect set-time, compressive strength, bond strength, shrinkage, and other desired properties. In various parts of the world, this type of material is also industrially known as "alkali-activated cement" or "inorganic polymer concrete" (Davidovits, 2011). Geopolymers provide comparable or better performance to traditional cementitious binders in terms of physical properties such as compressive or tensile strengths (Bell, 2008) (Buchwald, 2006) but with the added advantages of significantly reduced greenhouse emissions, increased fire and chemical resistance, and reduced water utilization (Alonso, 2001). The use of geopolymers in modern industrial applications is becoming increasingly popular based on both their intrinsic environmental as well as performance benefits. Historically, trial applications of geopolymers were first used in some concrete applications by Glukhovsky and co-workers in the Soviet Union post WWII; the geopolymer was then known as "soil cements" (Davidovits, 2011). Figure 1 shows a typical aluminosilicate structure that is common among many geopolymer materials.

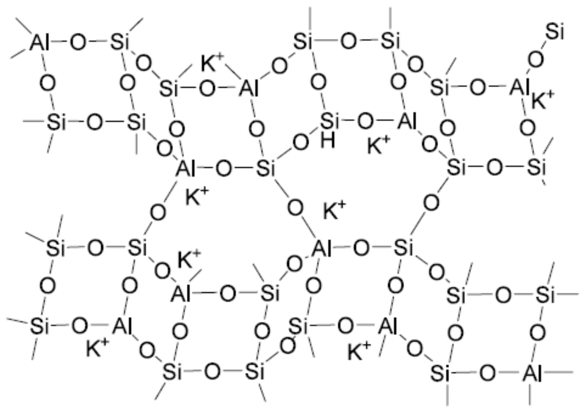

Figure 1: Example Aluminosilicate Molecular Geopolymer Structure

The structure of a geopolymer is a cross-linked inorganic polymer network consisting of covalent bonds between Aluminum, Silicon and Oxygen molecules that form an alumniosilicate back bone with associated metal ions. While any specific geopolymer structure, such as the one represented here in Figure 1, will be significantly more complicated based on the chemical make-up of the starting raw materials, the generic structure shown provides an excellent representation of how a geopolymer network is constructed. In contrast, OPC is a hydrated complex of small molecules that are not covalently bonded but rather associated. This is shown in a simplified structure in Figure 2. OPC itself is sufficiently complex that the structure shown in Figure 2 is only a basic representation of the molecules but no long chain covalently bonded backbone or network structure exists in standard cementitious materials.

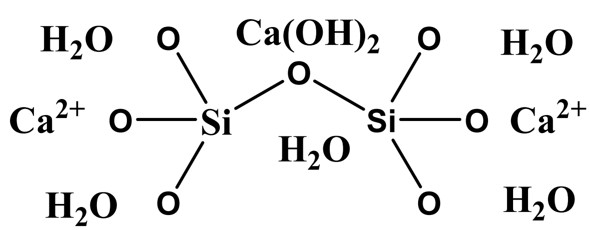

Figure 2: Simplified Example Molecular Structure of Hydrated Portland Cement (OPC)

3.0 GEOSPRAY GEOPOLYMER MORTAR

A specific example of a formulated geopolymer repair mortar is GeoSpray produced by Milliken Infrastructure Solutions, LLC. It is formulated to meet all the physical and chemical requirements for rehabilitating sewer and storm water structures. Water is added to the geopolymer at the job site where it can simply be centrifugally sprayed inside an existing structure that has been properly prepared. The exact formulation of most products are considered trade secrets, but generally speaking, geopolymers contains a mixture of the standard materials that are used in the production of calcium-aluminosilicates. Other components include, but are not limited to, blast furnace slag, reactive silicas, metal oxides, mine tailings, coal fly ash, metakaolin, calcinated shale, natural pozzolans, and natural/processed zeolites. Additional bio-based admixtures are included in the formulation in order to allow the composite material to set-up quickly and easily hydrate with a single addition of water. The "just add water" aspect of this particular geopolymer system has been specifically developed to avoid the typical alkaline activation mechanisms and order of addition complexities of traditional geopolymers which have limited significantly the ability of most contractors and asset owners from using geopolymers commercially. A summary of the physical properties of GeoSpray as compared to conventional concrete pipe repair mortars is included in Table 1.

Table 1. Typical properties of GeoSpray compared to Conventional Cement Based Repair Mortars

Test Method	Duration	GeoSpray	Conventional Repair Mortar
Compressive Strength ASTM C-39/C-109	1 Day 28 Days	Min. 2,500 psi / 17 MPa Min. 8,000 psi / 55 MPa	 5000 psi / 34 MPa
Flexural Strength ASTM C-78	7 Day 28 Days	900 psi / 6.2 MPa 1300 psi / 9 MPa	 500 psi / 3.4 MPa
Modulus of Elasticity ASTM C-469	1 Day 28 Days	3,000,000 psi / 20700 MPa 5,800,000 psi / 40000 MPa	 3,000,000 psi / 20700 MPa
Bond Strength to Concrete ASTM C-882	1 Day 28 Days	Min 1,300 psi / 9 MPa Min. 2,500 psi / 11 MPa	N/A
Set Time ASTM C-807 Initial Cure Time	Initial Set Final Set	60 - 75 Minutes 90 - 110 Minutes	120 Minutes 300 minutes
Freeze Thaw Durability ASTM C-666	300 Cycles	100% Zero loss	80% to 90% 10% to 20% degradation
Shrinkage ASTM C-1090	28 Days	0.00% @ 65% R. H.	0.35% to 0.50% Shrinkage
Tensile Strength ASTM C-496	28 Days	Min. 800 psi / 5.5 MPa	400 psi / 2.7 MPa
Abrasion Resistance ASTM C-1138	6 Cycles @ 28 Day Maturity	0.67% Loss	5.60% Loss
Rapid Chloride Ion Permeability ASTM C-1202	28 Days	Very Low	N/A

With this type of repair mortar the entire system is contained within original powder formulation, allowing a single step addition. It is common for these materials to be pumped up to 500ft within a pipe and still be centrifugally cast without clogging or damaging nozzle performance. To achieve this standard of performance, traditional cement or geopolymer formulations would require much higher water ratios which would degrade their ultimate strength and require a much thicker final product during the installation to meet the flexural strength requirements of the rehabilitation.

4.0 GEOPOLYMER ADVANTAGES

4.1 Cold Joints

On real world construction sites, unexpected and unanticipated circumstances can result in delays or work stoppages. Additionally, many job sites can be subject to restricted work hours due to local traffic issues or community related ordinance. When working with the placement cement, these types of work stoppages or delays can result in the formation of a cold joint. A cold joint is an undesired discontinuity between two layers of concrete. A cold joint occurs due to the inability of a freshly poured wet cement to intermingle and bind with an already hardened cement. A typical cold joint in a poured structure is shown in Figure 3.

Figure 3: Typical Cement Cold Joint

Cold joints can resulting in multiple problems ranging from minor to catastrophic. The spectrum of resulting issues include: minor cosmetic visual differences between layers, possible moisture intrusion into the joint resulting in degradation from environmental conditions, and areas of significantly compromised strength within a structure. When water is mixed with Portland cement (OPC) the cement reacts with the water to form a hydrate allowing the cement to harden around aggregates and form concrete. The chemistry of the reaction uses a hydration mechanism to create a hardened solid phase structure. However, once the hydration is complete and the structure is solid, it will not physically or chemically intermingle with additional cement.

Geopolymers undergo a completely different set of reactions classified as condensation. This process creates large polymer molecules that react to form large chain molecules that create the solid structure. When a hardened geopolymer is contacted with a freshly poured geopolymer mixture the polymer molecules from the hardened geopolymer are still active and will chemically bond with the new mixture preventing a cold joint from forming.

To demonstrate the superior properties of geopolymer mortar as compared to OPC materials with respect to cold joints, a series of compression test were conducted using 2 inch by 4 inch cylinders using a commercial geopolymer formulation

On the first day of the experiment, full cylinders of both geopolymer and commercially available competitive material based on OPC, both designed for use in structural pipe repair, were poured. In addition to the full cylinders, ½ pours of the same size were produced with both materials and vibrated on a slant to create an approximately 45º angle in the lower portion of the cylinder (as shown in Figure 4). A second pour atop the first pour (of the same material) was then done with intervals of 1, 7, 14 & 28 days. All samples where then compression tested according to ASTM C39.

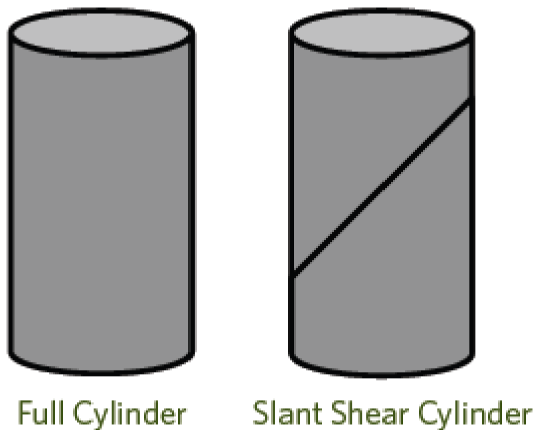

Figure 4: Schematic Illustration of Cold Joint Compression Experiment

For all combinations, the full cylinders poured on day 0 have no joints and break in a standard compression failure throughout the cylinder. For the geopolymer samples with the 45º joint, compression failure mode is the same as the full cylinder even when 28 days have elapsed between pours. The leading OPC competitive material breaks along the cold joint in all of the test intervals, showing that the cold joint formed in the OPC between the pours is the weakest part of the structure. Detailed images of the experiments are shown in Figure 5.

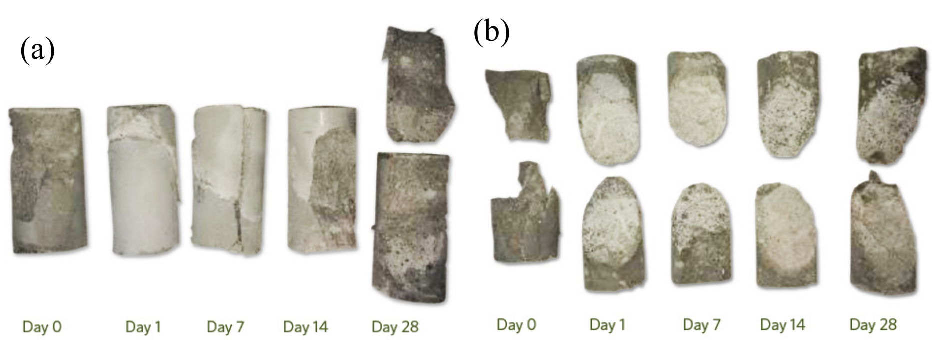

Figure 5: (a) Geopolymer samples showing compression failures located away from the joint (b) OPC samples with compression failure located at the joint.

4.2 Chemical Resistance

In sanitary sewers and other wastewater environments, the general corrosion mechanism of cementitious based materials is well known and widely documented. It is often referred to as Microbial Induced Corrosion or (MIC). The process of MIC involves a 3 step mechanism (shown schematically in Figure 6):

- First, hydrogen sulfide gas (H_2S), commonly referred to as sewer gas, is released by the reduction of sulfates in the sewer effluent from anaerobic bacteria – generally living in a "slime layer" below the water line.
- Secondly, sulfuric acid (H_2SO_4) is formed on exposed surfaces through the oxidation of H_2S by aerobic *Thiobacillus* bacteria.
- Finally, the sulfuric acid reacts most often with calcium hydroxide $Ca(OH)_2$ found in many cements to form gypsum $CaSO_4 \cdot 2H_2O$ which is water soluble and will wash away.

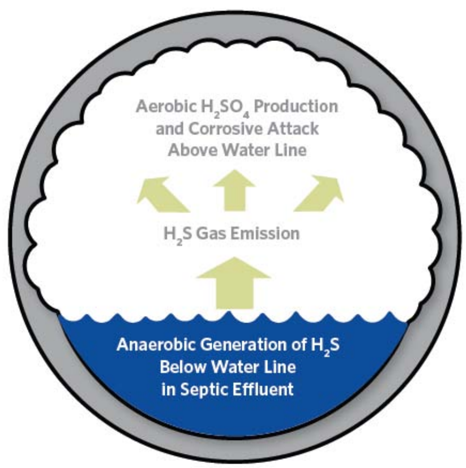

Figure 6: Schematic of the Chemical Processes associated with MIC Corrosion

The chemical make-up of geopolymers makes them inherently more acid resistant to the MIC mechanism found in many sewer environments. Geopolymers (dependent on the exact formulation) will contain greatly reduced concentrations of $Ca(OH)_2$ (calcium hydroxide) essentially preventing the acid corrosion mechanism found in many typical cements. Chemical resistant studies were performed following the procedures of ASTM-C267. Geopolymer sample cubes were cast and allowed to cure for 28 days before being soaked in both water and 7% sulfuric acid (pH 0.9). OPC cubes were also cast and soaked as representative samples for standard reinforced concrete pipes commonly found in sanitary sewer systems.

Samples were measured for weight and dimensional changes after soaking for 1, 7, 14, 28, 56 and 84 days. 3 samples of the materials were soaked and tested, and the solution volume relative to the cubes was held constant. The chemical solutions were refreshed on day 14, 28, and 56. Geopolymer samples showed only slight loss of mass and signs of surface corrosion through the 84 days exposure to 7% H_2SO_4 (sulfuric acid), while the Portland cement samples lost more than 50% of their weight over the same time period. Figure 7 shows samples cubes before and after 84 days of soaking exposure.

Figure 7: Image of cubes before and after soaking in 7% H_2SO_4

Figure 8 shows the effect of the 7% sulfuric acid on weight of the geopolymer and OPC cubes over the same time period. The results of weight are normalized to the percentage of weight change of samples soaked in water to account for the absorption of water. Through the 84 days exposure the geopolymer corrosion was approximately 1/5th of the standard OPC material.

When tested under the ASTM C-267 protocol against aggressively corrosive 7% sulfuric acid (pH 0.9), the geopolymer showed only approximately 5-7% (note: the samples are compared to water soaked materials and the below 0 starting point is due to gel formation of H_2SO_4 and not true weight loss)weight loss and slight surface corrosion compared to the >50% weight loss observed in OPC samples that reflect concrete sewers in use today. Where concrete pipes and structures exhibit the effects of microbial induced corrosion, geopolymers should result in significant resistance improvement over OPC.

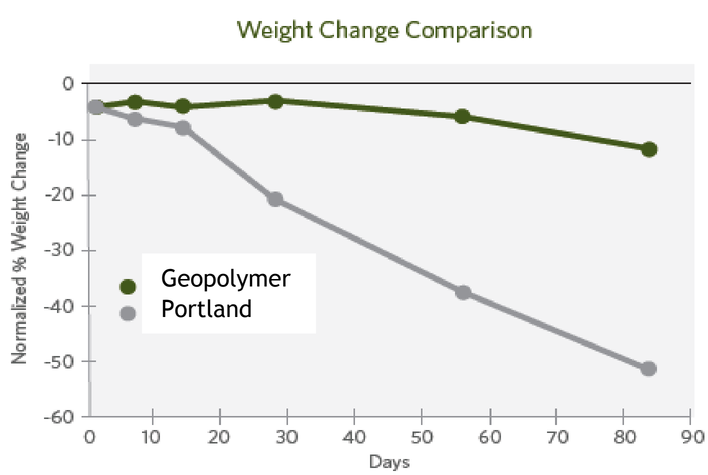

Figure 8: Comparative weight change over time for soaked cubes

5.0 CASE STUDIES

5.1 McALLEN, Texas (114 inch CMP Storm Drain Rehabilitation)

In the rapidly growing Texas border town of McAllen, dealing with storm water runoff is a challenge. The weather fluctuates quite rapidly, and ensuring that storm water infrastructure is capable of dealing with large amounts of water quickly is of paramount concern. As the population of the community has nearly tripled over the past two decades some of their storm water infrastructure has presented an ongoing challenge.

Such a problem was the Rado Storm Drain, located within one mile of the Rio Grande river. The storm drain consists of 2 side by side 114 inch corrugated metal pipes (CMP) each over 2200 linear feet in length. The pipes had issues ever since they were installed and have been repaired in various sections over the past decade with a non-structural shotcrete and a bitumen coating. These attempted repairs were done over short segments of the pipe, but large scale separation of the joints along with water infiltration continued to be major concerns.

The local municipalities had experience with non-structural repairs in the past that had not been successful on this particular application. They investigated several repair options including Cured-in-Place-Pipe (CIPP), slip-lining, and geopolymers. Both the CIPP and slip-lining solutions were significantly higher cost with additional complications due to the large diameter. The community decided to specify a cementitious lining as their structural application. In addition, because the county was a member of HGAC Buy (Houston-Galveston Area Council), a competitively bid contract organization with members in 48 states, they were able to specify geopolymers and avoid the costly process of bidding the project themselves. Inland Pipe Rehab, LLC using their Ecocast™ process installed GeoSpray as the repair solution for this project in the spring of 2014.

Because there were two side-by-side pipe sections, only one was repaired at a time and all of the flow was diverted to the other section. Each pipe section was cleaned and inspected for joint failure, cracking and infiltration. Stopping water infiltration was the primary challenge of the project and required meticulous preparation. These issues were addressed with hand repair to ensure that all the infiltration of water was stopped and a continuous surface for the application of the geopolymer mortar was created. Once these issues were tackled, 150 to 300 foot sections were then sprayed with the final engineering designed thickness of 1.5 inches of geopolymer. The ability to apply a 1.5 inch thick layer in a single spray pass saved both time and cost for the asset owner. During most days of operation, the contractor was able to apply between 20,000 and 40,000 lbs of geopolymer in a single run within the pipe, creating a truly monolithic structure.

The use of geopolymer to create a new pipe within the existing CMP structure that existed was completed on time and under budget. The new pipe is ready to handle the unpredictable storms of southern Texas for years to come. Figure 9 shows a series of images from the job site, this includes upstream the entrance to the pipes, a view of the joint separation on the shotcrete repaired structure, the geopolymer application and the finished pipe.

Figure 9: Images from the jobsite for the Rado Storm Drain in McAllen, TX

5.2 Cincinnati, Ohio (36 inch Sanitary Sewer Rehabilitation)

During an assessment of their combined sewer system, the Metropolitan Sewer District of Greater Cincinnati (MSD) discovered that an 800 ft section of stacked stone and brick sewer line, situated in an area with active natural springs, was badly leaking. Records indicated that the old stone pipe was likely installed in phases between 1870 and 1890.

This specific section of stone pipe presented a host of unique challenges. One section of the pipe was on a 26° slope that lead to a river outfall. A detailed inspection revealed that the pipe had both round and arched cross sections with two different diameters. A first 500 ft section had an inner diameter of approximately 60 inches, and a second 300ft section had a 36 inch diameter section. Access to the pipe itself was also a challenge. The old sewer ran beneath three sets of road crossings, all at different elevations. It was also located near the University of Cincinnati campus, so minimizing surface disruption was a high priority.

In reviewing their options to address the deteriorated pipe, the MSD quickly realized that replacement was not a viable option because of the pipe's depth and location. When considering alternative trenchless methods a number of options were evaluated including Cured-In-Place –Pipe (CIPP) and Slip-lining. CIPP was not a viable solution due to the variation of the pipe's different shapes and sizes, the steep slope and the rough protruding stone. Slip-lining of the pipe was also a poor option for the same reasons, would have required digging several large access pits, and resulted in significantly reduced flow capacity. In the end, the MSD chose to apply a geopolymer lining that would be both hand and machine sprayed to create a new structure lining, repair the leaking, and return the pipe to its original shape.

Construction on the site began in March 2012.The first task was to clean the stone pipe with a high pressure wash and then to use a hand spray application of geopolymer to stop the leaking and to stabilize the existing stone structure. One critical advantage of the technique of centrifugally spraying a geoplymer liner is that the equipment foot print can be limited to the size of approximately two 24 ft box trucks and spraying can occur more than 400 ft from the actual mixing location . This allowed the crew to avoid any traffic disruption.

Once the initial hand spray was complete, a mechanical sled system was used to apply the final coats of the geopolymer and arrive at the engineer's required thickness allowing for a new structural pipe to be built within the existing pipe. It is interesting to note that not only did the pipe itself present many challenges, the weather also was a key consideration. The temperature above ground during the installation period ranged from just above freezing to highs in the mid 70s (°F). These large temperature swings are no problem for geopolymer systems, helping keep the project on schedule. From start to finish the full project was completed in under six weeks, ahead of schedule and on budget. The flexibility of geopolymers makes them an excellent choice for the toughest sewer repairs. Figure 10 shows a series of images from the job site.

6.0 CONCLUSIONS

Geopolymer mortar repair systems have been developed to be a cost effective alternative to other trenchless repair systems for large diameter pipes. Geopolymers have advantages over traditional OPC systems relating to the chemistry of the materials and how they are reacted that include (a) lower CO_2 footprints, (b) reduced tendency for cold joints and (c) enhanced chemical resistance. Multiple case studies have been shown where structural pipe repairs were designed and completed for both storm drain and sanitary sewer applications.

Figure 10: Images from the jobsite for the MSD in Cincinnati Ohio

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An Evaluation of Trenchless Point Repair Solutions for Pipes of Varying Inner Diameter and Offset Joints

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Abstract

There are numerous point repair solutions in the pipeline industry. These can be broadly grouped into internal joint seals, mechanical sleeves, CIPP short liners and other. The pros and cons of available methods are not always clearly understood by the owners, engineers and installers. Some widely known brand name manufacturers of joint seals and mechanical sleeves are: WEKO-SEAL®, In-Weg seal, Quick Lock, Link-Pipe, LMK, HydraTite, Amex 10, and other. These products can be used for storm, potable water, wastewater and industrial pipes, conduits and drainage culverts. Materials for these types of repairs are specified based on anticipated exposure conditions after installation. When owners are in need of an economical solution for the maintenance of pipelines, the point repair solutions present an attractive alternative to more extensive repair or replace options. Quick Lock that meets ASTM F3110 is new in North America although it has been used for over 20 years in other countries. This paper introduces its features that no other point repair solutions offer. In addition, given that the suitability of the technology for a particular application shall always be jointly decided by the owner, the engineer and the installer, this paper provides an evaluation of available methods during this decision making process.

INTRODUCTION

Based on the trends in the market studies the second author has performed for a number of clients over the past three decades, more than 3,000,000 mechanical sleeves and internal joint seals have been installed globally for repairing the following types of defects in pipe: longitudinal, radial and circumferential cracks, fragmentation, leaking joints, displacement or joint misalignment, closing or sealing unused laterals, corrosion, spalling, wear, leaks in the barrel of the pipe, deformation in the pipe and root penetration. Repairs can be made to vitrified clay, concrete, reinforced concrete, plastics, glass reinforced plastics, cast iron, ductile iron and steel. When owners are in need of an economical solution for the maintenance of pipelines, the point repair solutions present an attractive alternative to more extensive repair or replace options.

Mechanical trenchless repair sleeves with a locking gear mechanism for pipes of varying inner diameter and offset joints in the range of 6 to 72 in (150 to 1800 mm) offer many advantages over relining the entire pipe or using other point repair technologies. These sleeves can be used for storm, potable water, wastewater and industrial pipes, conduits and drainage culverts. Given that each owner retains the right to choose the test protocol to verify the efficacy of these sleeves to provide a leak free repair, the producers have not provided any test protocol of their own - except for internal joint seals. The maximum internal pressure this sleeve can carry depends on the diameter and the wall thickness, ranging from 145 to 217 psi (1.0 to 1.5 MPa); the external pressure cannot exceed 21.7 psi (0.15 MPa) as hydrostatic groundwater pressure acts in a manner which causes the integrity of the compression seal to be compromised. It should be noted, however, that it is the pressure differential that must be determined. The external pressure must exceed the internal pressure by 21.7 psi to result in leakage; ultimately, the worst case scenario is when the pipe is empty with high groundwater.

More than 200,000 Quick Lock sleeves have been installed globally for repairing the following types of defects in pipe: longitudinal, radial and circumferential cracks, fragmentation, leaking joints, displacement or joint misalignment, closing or sealing unused laterals, corrosion, spalling, wear, leaks in the barrel of the pipe, deformation in the pipe and root penetration. There are no limitations on the diameters of the laterals that can be sealed. The degree of pipe deformation that can be repaired is dependent on the minimum and maximum diameters for which the sleeve is applicable, up to 5% deflection is acceptable. Repairs can be made of vitrified clay, concrete, reinforced concrete, plastics, glass reinforced plastics, cast iron, ductile iron and steel. The suitability of the technology for a particular application shall be jointly decided by the owner, the engineer and the installer. For example, all materials in contact with potable water are certified to meet National Sanitation Foundation/American National Standards Institute (NSF/ANSI) 61/372. When the materials for the mechanical sleeves or joint seals are selected to meet the project demands and installed in accordance with the ASTM standard F3110-14, the renovation extends over a predetermined length of the host pipe as a continuous, tight fitting, leak free, and corrosion resistant repair.

There are other mechanical repair sleeves and joint seals and the details are presented in later sections. Then we have short sectional liners. The extent to which these short liners could offer structural augmentation for withstanding internal and external pressures is lower compared to that from the use of mechanical or internal seals, because of the dramatic difference in the engineering behavior of the materials used between the two types of methods. These are also cumbersome to use and time consuming to set up, prepare and install. Health of the workers installing them is also more at risk.

COMPONENTS FORMING THE SLEEVE

A Stainless steel sleeve with flared end, faces the direction of flow and improves the hydrodynamics, prevents solids from depositing, and increases jetting resistance shown as (1) in Figures 1 and 3. Flared ends are not used, however, for potable water applications; Metal overlap is for expanding to the pipe wall shown as (2) in Figure 1; The locks that keep the sleeve expanded run along the toothed strip shown as (3) in Figure 1; The lock is a small set of gears that only moves in one direction, thus keeping the sleeve expanded shown as (4) in Figures 1 and 3. The locking gears are also shown in Figure 4. There are three sprockets per gear lock. Two of the sprockets in the gear ride in the corresponding "teeth" in the sleeve. The third sprocket is the lock. It allows the other two gears to only move in a forward direction. The gear and the shield are all of the same material as the rest of the sleeve. Furthermore, the gears are protected by a cover to prevent snagging of waste, build up of sludge or sediment, and protect it from a root cutter and cleaning nozzle. These features have a proven track record of over 20 years. Adhesive tape and plastic pin are put on at the factory to protect the sleeve during transport and prevent it from unrolling shown as (5 and 6) in Figure 1; circumferential seals are formed from the rubber being compressed against the host pipe.

For single installations, the damaged section must always be between the sealing knobs shown in (7) in Figures 2 and 3.There is a trimming line marked in the rubber jacket. It shows the installer where to cut off the projecting rubber end shown as (8) in Figure 2. For serial installations, the projecting rubber end is not cut off and acts as a seal between the sleeves installed shown as (9) in Figures 2 and 3. There are designs with fewer sprockets and made of materials other than that of the body of the sleeve with a record of about a year – somewhat experimental.

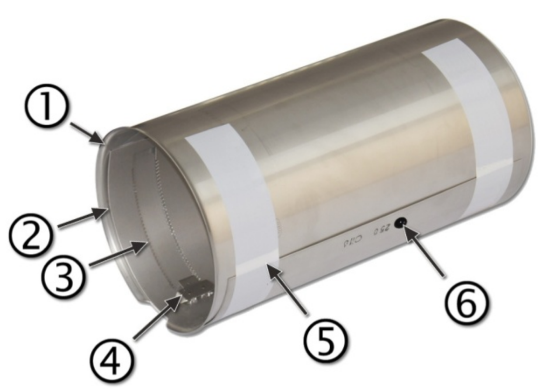

Figure 1. Stainless steel sleeve (Uhrig, 2015)

Figure 2. Locking gear mechanism and EPDM rubber seals (Uhrig, 2015)

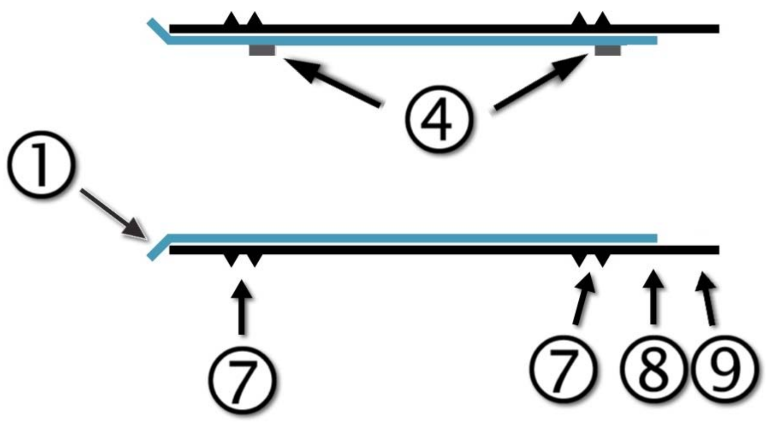

Figure 3. Cross sectional view of the sleeve and the EPDM seals (Uhrig, 2015)

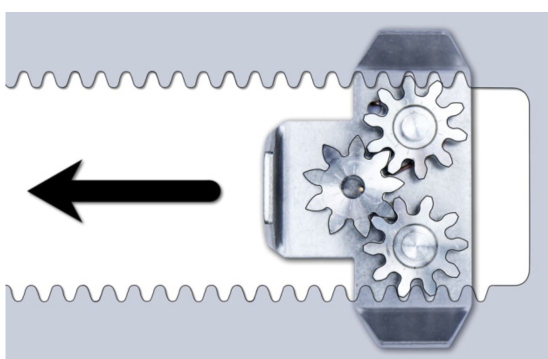

Figure 4. Locking gear mechanism (Uhrig, 2015)

INSPECTION AND PREPARATION OF DAMAGED PIPE

Before using a point repair solution, the pipe must be inspected to ascertain whether it can be repaired with the system. Cleaning and inspection shall be as per National

Association for Sewer Service Companies (NASSCO) guidelines for sewage pipes. For other pipes, cleaning and inspection methods that are acceptable to the authority shall be chosen by the installer. There must be at least one access via a manhole or an inspection chamber. The manhole must have a diameter of at least 24 in (600 mm) so that the camera/packer system can be inserted. The interior of the pipeline shall be carefully inspected to determine the location of any condition that shall prevent both deployment and proper installation, such as large joint offsets, roots, and collapsed or crushed pipe. The pipe to be repaired must always be cleaned with a high-pressure jet, a drag scraper or an equal approved by the owner, the owner's representative or the manufacturer, before using the sleeve.

MATERIALS

The sleeve is made of high grade 316 or 316L stainless steel per ASTM 240 (2015) and ASTM 666 (2010). There are three types of sleeves available: non-flared, one end-flared and both ends flared. Non-flared sleeves are mainly used for potable water applications or serial installations, followed by one end-flared sleeves. The both ends flared type is used for single installations only. Ethylene Propylene Diene Monomer (EPDM) seal or silicon rubber seal (for potable water with chlorine or fluoride treatment or even EPDM washed with peroxide) meets the physical property requirements for elastomeric materials used in cold water supply, drainage, sewerage and rainwater systems for Type WC in Table 2 of EN 681-1 Hardness Category of 40. Nitrile Butadiene Rubber (NBR) and similar elastomers may be appropriate for other effluents or where contaminated soils are surrounding the exterior of the pipe.

INSTALLATION

In pipes up to 32 inches in diameter (800 mm) the sleeves are installed using an inflatable packer on wheels. The packer is usually connected to the camera or robot via a bracket and a hollow link bar. For repairs more than 20 ft (6 m) into the line the sleeve is installed most efficiently and accurately when the packer is pushed or pulled and positioned by using a crawler camera, equipped with an accurate distance counter. The sleeve is usually positioned on the packer while in the manhole. Once the packer/sleeve is in the correct position over the damaged section, the actual installation takes only about 1 to 2 minutes. If the installer is using a packer equipped with a laser, position the sleeve so that the laser beam is reflected both on the edge of the sleeve and the host pipe. Installation in pipe sizes larger than 32 inches (800 mm) is done manually. Depending on the quantity of flow present, bypassing in accordance with the authority's requirements may be necessary.

OTHER MECHANICAL SLEEVES

a) Link-Pipe asserts that if during routine inspection damaged pipe is discovered, it takes around 20 minutes to have it repaired using Link-Pipe Grouting Sleeve or SewerSealer, shown in Figure 5. These sleeves must be installed before the resin is cured, usually within 20 to 25 minutes. Spot repairs can be installed in pipe diameters of 6-54 inches. Every Grouting Sleeve, SewerSealer Sleeve and Link-Pipe (PVC Sleeve) carries a manufacture 10-year Limited Warranty and is evaluated to give a minimum 100-year service life. Link-Pipe contains a wide variety of diameters and ranges from man accessible to remote repair products. The sizing of the sleeve that would work needs to be established with precise measurements of the inner diameter of the pipe that is being repaired. At times, this lack of the sleeve's ability to accommodate even minor variations in the diameter of the damaged pipe presents challenges in the field.

Figure 5. Sewer seal (Link-Pipe, 2015)

b) WEKO-SEAL®'s advantages over conventional pipe-joint repair methods are, non -corrodible, bottle-tight seal with minimal reduction of the pipeline's interior diameter; operating pressures in excess of 300 psi and 100 feet of external head pressure, with proper design, accommodates normal pipe movement from ground shifting, thermal expansion or contraction, and vibration; it is patented technology with a positive mechanical locking wedge design, test valves standard in all seals, durable cross-sectional seal thickness. Design and installation options for standard round pipes as well as lines with unusual shapes including oval, square or those having compound angles are feasible. Installation with access openings can be in excess of 2,000 feet apart; entrance through manholes, vaults, fittings or cut-outs, with fast installation and minimal lead time, the technology lends itself to emergency situations.

Over 40 years of turnkey installation experience with 300,000 seals installed, Miller claims that they are the most experienced contractor in the industry. All installations are warranted. Miller Pipeline's trained, experienced personnel handle the seal installation process in full compliance with OSHA's 29 CFR 1910.146 Permit-Required Confined-Space regulations with full rescue and retrieval equipment on-site at all times. Miller's crews are also trained in CPR, SCBA, First Aid and emergency rescue procedures.

The material specifications for the WEKO-SEAL® fall into four main application categories: potable water, wastewater, natural gas and seawater/brackish water. Each application has materials specifically engineered to provide years of worry-free maintenance through the proper rubber seal and stainless steel retaining band selections. Seals designed for use in potable water applications are made from EPDM rubber seal and type 304 stainless steel retaining bands. All components used for potable water applications are NSF 61 drinking water approved by Underwriters

Laboratories. Storm water installations use the exact material components as identified for potable applications. Wastewater applications use the EPDM rubber seal shown in Figure 6 and type 316/316L stainless steel retaining bands. Seawater/Brackish Water Installations in salt water environments consist of an EPDM rubber seal and AL6XN retaining bands. Natural Gas Seals designed for natural gas applications are constructed of Nitrile butadiene acrylonitrile rubber. In areas where pipelines are likely to be exposed to petrochemicals or where oil resistance is of concern, Nitrile or Neoprene rubber gaskets can be considered as an option to EPDM rubber.

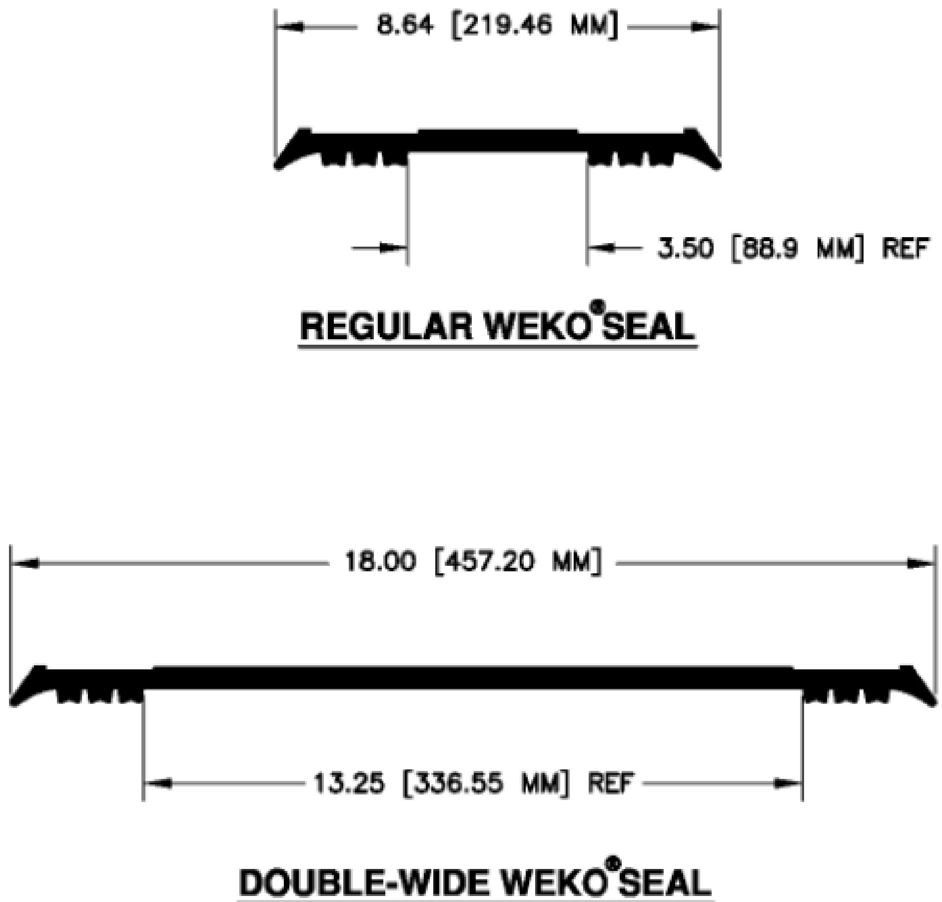

Figure 6. Samples of weko-seal (Miller, 2015)

c) Creamer In-Weg® seals are designed for the internal sealing of leaking pipe joints or cracks in all types of pipe materials, including cast iron, ductile iron, concrete, reinforced concrete, steel, vitrified clay and plastic piping systems having pipe diameters of 16 inches or larger. These seals permanently eliminate leaks at internal pressures up to 300 psi. Excavations for access can be as much as 5,000 feet apart, causing very little disruption to traffic and greatly reducing restoration cost. Over 50,000 seals have been successfully fitted worldwide.

d) HydraTite Internal Joint Seal is a mechanical, trenchless remediation for repair of pipe joints as shown in Figure 7. The HydraTite system consists of a proprietary rubber seal which spans the joint and is held in place by stainless steel retaining bands in either side of the joint. These retaining bands are expanded and locked in place using a wedge design which forms an air tight clamp around the joint eliminating all infiltration and exfiltration. Each HydraTite seal is designed and custom made for the application to ensure complete compliance with project specifications. The HydraTite System is a recognized method of joint repair by AWWA manual M28 (2001).

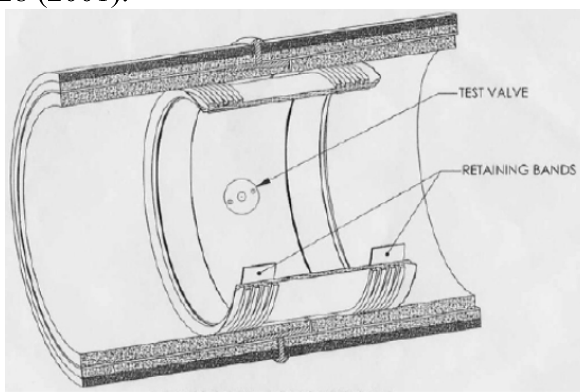

Figure 7. Joint seal (HydraTite, 2015)

e) AMEX 10 offers internal mechanical sleeves for pipes under the brand names MONO, VARIO, LEM and SPEED in the range of man entry sizes up to 240 inches for internal pressures up to 25 bars and external pressures up to 8 bars. The sleeves have been used in broad applications worldwide for over 30 years. In fact, Amex 10 and Miller had a working arrangement for some years until both decided to pursue their own future plans. Uhrig that offers Quick-Lock, Quick-Lock Big and Liner End Seal and Amex 10 have a working arrangement at the present time. Some of their efforts are focused on developing jointly a new product line to meet changing market needs.

f) For Snap Lock repair sleeve shown in Figure 8, no chemicals are needed for installation. Made of high-grade stainless steel and surrounded by a rubber outer sleeve, the snap-lock system is highly durable and resistant to most chemicals including hydrogen sulfide. The installation process locks the Snap Lock© module permanently in position. The Snap Lock© repair module is comprised of a cylindrical stainless steel sleeve surrounded by an outer sleeve of rubber incorporating a specially designed seal. The outer sleeve of buadienstryene rubber is also chemically resistant. Hydrophilic rubber bonded to the outer sleeve expands up to 300% in the presence of water, ensuring a completely watertight repair for site conditions where water is present consistently.

CURED IN PLACE PIPE (CIPP) SHORT SECTIONAL REPAIRS

There are many providers of these repairs: Easy Liner LLC, Flow-Liner, Formadrain®, Magnaline, Master Liner, Nu Flow Technologies, National Liner, Perma-Liner, Reline America, Stephen's Technologies, and LMK Technologies. CIPP lining system for short patch repairs is used to fix cracks, holes, stop leakage, displaced/failed joints, prevent root intrusion, and help maintain the integrity of the existing pipe. Installation procedure is composed of these steps for CIPP short liners:

1. Locate/identify problem w/TV inspection camera system.
2. Clean and remove debris.
3. Assemble packer w/fiberglass mat & resin.
4. Install assembled packer into sewer line.
5. Center mat over damaged area in pipe.
6. Inflate the packer to 25 to 30 psi until liner touches and cures.
7. Force the resin to migrate into the damaged area(s)
8. Allow resin to cure for 2 to 3 hours.
9. Deflate packer & remove from pipe.

Figure 8. Snap lock system

Typical applications:

- 3" to 36" pipe diameters
- Municipal, commercial and residential uses
- Repair cracked pipes
- Offset and open joint repairs
- Bridging of missing pipe

Advantages of sectional liner repair systems

- Fast ambient cure
- Structural repair of pipe without excavating
- Non-disruptive to traffic
- Corrosion resistance
- Cost effective
- Consistent & uniform thickness
- No by-pass pumping
- Fast and efficient installation
- Helps eliminate future water infiltration and root intrusion

a) Master Liner’s Sectional Pipe Renewal System offers a solution for spot repairs in a matter of hours. Features include improved flow characteristics; high strength, leak proof epoxy; and custom diameters and wall thickness.

b) Logiball offers a complete line of reinforced sleeve installers and carriers for sectional liners (CIP or mechanical locking sleeves) up to 50 feet long for 4- through 36-inch pipes. In their shorter version, the Sleeve Installers are used as end plugs to process manhole-to-manhole liners (deformed, reshaped, expanded and CIPP liners). The Sleeve Installers are made of a two-ply, cross-biased reinforced rubber sleeve that is resistant to hot water and even steam under pressure. The rubber sleeve is secured to the end plate through a series of wedging points for a strong and safe attachment even when the rubber is softened by exposure to high temperatures.

c) LMK Technologies’ Performance Liner Sectional system is a unique one-step air-inversion point repair process and the only system that is compliant with ASTM F2599-11 which uses swelling gaskets of hydrophilic rubber. The liner is vacuum impregnated, meaning it is clean and safe for workers and the environment. Furthermore 100% resin migration is achieved at the point of repair as described per ASTM in F2599-11. Another advantage to this water-tight, structural system is that the liner is inverted through the point of repair versus pushing or pulling the impregnated liner through the damaged section. These liners are also properly undersized for the diameter of the host pipe and provide a finished product that complies with the design thicknesses per ASTM F1216. These properly undersized liners will invert at 2 to 3 psi with a holding pressure of 4 to 7 psi. This system renews mainline diameters from 6 to 42 inches. Smaller diameter (6 to 12-inch) sewer pipes can range in length from 3 to 100 feet. Larger diameters can range from 3 to 30 feet. Typically the resin is cured within two hours at ambient temperatures, or as fast as 30 minutes using LMK's steam curing system. Table 1 provides an evaluation of the Performance Liner against others.

d) The Perma-Liner Point Repair System consists of a fiberglass reinforced liner and ambient cured resins. The Point Repair Kits are sold from 6 to 54 inches in diameter and lengths from 2 to 30 feet. The point repair forms a structural, permanent waterproof repair which seals all types of pipe against infiltration and exfiltration. This repair has a superior bond to the existing pipe in both wet and dry conditions and because the resin is 100 percent solids, there is no shrinkage and therefore no annular space between the pipe and repair that could cause leakage. The Perma-Liner Sectional Point Repair is considered a structural repair per ASTM F1216-09.

SUMMARY

The most suitable point repair system needs to be chosen by a careful consideration of the following factors: structural capability, hydraulic characteristics, design life, ease of installation, need to deploy human labor or can be done remote using a packer or robotic means, corrosion resistance, ability to stop leaks and root intrusion, need to use chemicals or can be mechanical, cost of materials, labor and maintenance, track record, and whether the technology has been vetted by one or more ASTM and NSF standards. In the final analysis of pros and cons of internal joint seals, mechanical sleeves, CIPP short liners and other point repairs, the market will always have a need for multiple products. Therefore, mechanical sleeves and sectional repairs are here to stay given the mantra "doing more with less," and it is only a matter of time that the total number of sleeves used in USA would become higher than all other countries around the globe in a given year.

ACKNOWLEDGEMENTS

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Table 1. Performance Liner versus Others (after LMK Technologies, 2015)

Factors to consider	The Performance Liner	Packer Wrapper Spot Repairs
Configuration	The tubular liner allows the worker to pour the resin into the liner tube. This method is clean because the resin is contained.	Resin soaked flat sheet liners are wrapped on a sewer plug and held with rubber bands, Velcro, zip ties, or other.
Liner Impregnation	Vacuum impregnation provides a thorough wet-out of the liner ensuring the tube is completely filled with resin, not air.	Open resin pouring is a messy process and is difficult to verify thorough wet-out. There is also no evacuation of air.
Resin Migration	Extra resin that is added to the liner will penetrate fractures and open joints in pipe.	Flat sheet liners can only carry the amount of resin that the liner will absorb.
Versatility	Diameters up to 42" and continuous lengths up to 50 ft thru' 22" manhole-liner/bladder assembly is collapsible.	Liners wrapped around a sewer plug are difficult to maneuver and limit the dia and length of the liner through manholes.
Installation	The liner is positioned at the point of repair and then inverted through the damaged section, never pulled through the damaged section.	Liners on a sewer plug are pulled through the damaged section with pipe pieces being snagged leaving a convoluted repair or even a total collapse.
Inflation Pressures	Liner/bladder is at 1-2 psi and a holding pressure 5-6 psi.	Sewer plugs are inflated at 28-35 psi damaging the pipe more.
Documentation	Inversion installation allows the installer to view the liner before, during, and after, so that the placement is exactly where the repair needs to be.	Wrapped liners are shorter than the plug and the liner cannot be viewed. Therefore, positioning is solely dependent on measuring.
Length of Repair	A continuous length liner provides uniform wall thickness over 3-50 feet.	Multiple short length liners that overlap one another leave inconsistent wall thickness.
Assurance	The resin is protected as the liner is carried to the point of repair.	Allows the resin to be contaminated and wiped off during the winching-in process.
Safety	This system is clean and environmentally safe because there is no exposed resin.	This system requires handling of exposed resin soaked liners by the workers.
Standards	Compliant to ASTM F2599-11.	Non-compliant to F2599-11

Effective Repair of Incidental Construction Damage to 54-inch PCCP Line

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Abstract

A water utility located in the Southern US provides water to over 16,000 customers and wastewater service to over 13,000 customers across two counties. In order to serve a growing population, the utility is in the process of upgrading its water treatment plant from 51 MGD to 62 MGD. As a part of the construction process, soil nails were installed for excavation on the property. During the implementation of these soil nails, two 6-inch diameter punctures were made in a 54-inch Prestressed Concrete Cylinder Pipe (PCCP) and 45-degree steel elbow respectively. Given that this pipeline is the main raw water line for the treatment plant, and that the damage occurred during peak summer demand, the need to repair the line was urgent. The proximity of multiple bends and steep slopes adjacent to the distressed regions of the pipelines posed significant challenges for replacement. After extensive research and a site visit to a large municipality pipeline project utilizing similar technology, the utility proposed an internal structural repair using carbon fiber. The owner's options analysis process, implementation of the bypass system, as well as the internal repair will be detailed in this paper.

BACKGROUND AND IDENTIFYING THE DAMAGE

In July 2014, a water municipality in the Southern US was in the process of a phased water treatment plant expansion project to accommodate the significant growth in the area it serves. While excavation work was being performed and soil nails were being installed to stabilize a slope at the job site, the Engineers noticed a leak started to take place on the slope. Initially the leak was thought to be groundwater from recent rains in the area. However, it was soon realized that this leak appeared to be from the raw water main in the adjacent area. To verify the source of the leak, plant personnel and the Engineers began conducting experiments. First, runoff water from the slope was directed to one channel for sampling chemicals and measuring flow rate. Second, chlorine was added to the intake location of the raw water line to test for residual in the slope runoff water. Also, the raw water pipeline's flow was ramped up and down to see if measureable changes in slope runoff could be documented. This testing determined that the source of the leak was the 54-inch PCCP raw water main, which transports water from the pre-sedimentation basin to the conventional filter flash mix tank at the utility's water treatment plant.

To investigate the damage and identify any sections of distressed pipe, a diver was sent into the pipeline to observe and take video. Two specific damage locations were identified through the video inspection. The diver's inspection noted two holes in the pipeline: one at the straight section of pipe directly upstream of a 45 degree bend in the pipeline and one within the 45 degree bend section in the pipeline as shown in Figure 1 below. Measurements of the hole diameters verified the sections of the pipeline had been punctured by soil nails and the associated 6-inch diameter pilot holes that were drilled into the pipeline (See Figure 2 and 3 below).

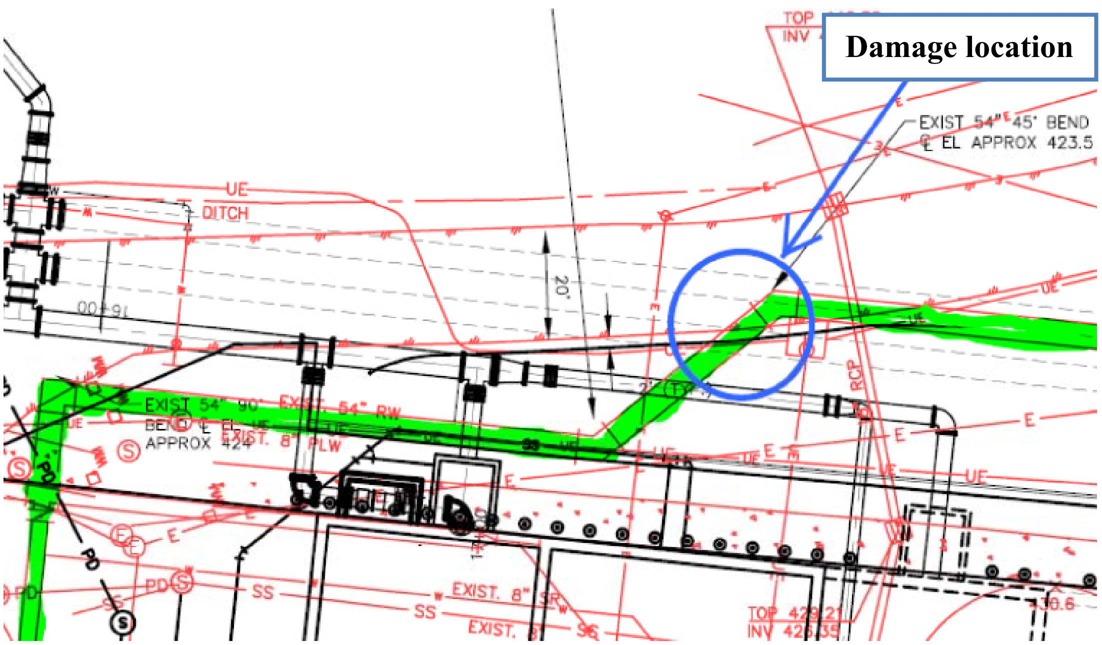

Figure 1. Location of distressed pipe sections within the pipeline (green highlighted line is pipeline repaired)

Figure 2. Soil nail going through pipe wall

Figure 3. Hole in steel elbow pipe section

OPTIONS ANALYSIS

Based on the damage identified by the video inspection, it was determined the penetrated sections of pipe could not be relied on safely for long term operation. The sections required either replacement or structural upgrade.

As part of the options analysis, replacement, slip-lining, and stand-alone structural rehabilitation with Carbon Fiber-Reinforced Polymer (CFRP) were considered. The proximity of multiple bends in the pipeline to the distressed pipes ruled out slip-lining as a repair option. Due to the pipeline being positioned adjacent to a 20 foot drop-off, excavation for replacement would have been extremely challenging (see Figure 4). Furthermore, lead time for replacement of the special order steel bend section would have left the pipeline unrepaired for an extended amount of time during the remaining peak summer demand. As the need to repair the line was pressing, the team decided that trenchless repair was the best option. Based on the notable external load carrying requirements and the construction schedule restraints, a CFRP lining system was selected as the repair method for its structural capabilities, trenchless application, and timeliness.

Figure 4. Location of pipeline in relationship to excavation slope

The CFRP lining system was designed as a stand-alone system without reliance on the host pipe for structural integrity. In addition to taking the internal pressure loads of the pipeline, the repairs were required to take into account all external loads including soil pressure and the vehicular load of a Manitowoc Model #555 crane. With a crane weight of 150,000 lbs and a load carrying capacity of 30 tons (60,000 lbs), this totaled to 210,000 lbs of vehicular load. The load is distributed onto two tracks, each 3ft by 21ft, which translates to approximately 1,700 psf load acting on the soil.

In addition to addressing the distressed sections, the adjacent sections also needed to be evaluated to determine whether they had also been structurally compromised. The owner considered involving a non-destructive evaluator to assess nearby pipe sections; if damage was found the adjacent sections would also require replacement or stand-alone structural rehabilitation. In the end, the owner decided to address the nearby segments assuming they were structurally compromised, and allocate funds directly to repair, in lieu of inspection followed by repair. Structural Group was then called in to conduct the repair.

Given that the owner had no previous experience with the use of CFRP, it was decided that a direct visit to an active CFRP upgrade project would be beneficial. The CFRP installation contractor had a project ongoing with Miami-Dade Water and Sewer Department (MDWASD) at that time repairing 54-inch PCCP, so an exact comparison would be possible. The CFRP installation contractor coordinated with Miami-Dade WASD and the owner was able to successfully visit the jobsite in South Florida, observing the CFRP installation process and helping to finalize the decision to move forward with CFRP repair.

IMPLEMENTATION OF THE BY-PASS SYSTEM

Given the critical nature of the affected 54-inch pipeline, the owner elected to set up a temporary by-pass upstream and downstream of the repair area, so the pipeline could remain operational throughout the repair process. As shown in figure 5, the by-pass system was set up on the Site prior to the repair process moving forward.

Figure 5. By-Pass System

IMPLEMENTATION OF CFRP INTERNAL REPAIR

The CFRP repair process involves layers of unidirectional carbon fiber fabric being installed in a pipe longitudinally and circumferentially. In areas with steel substrate such as the joints, a layer of glass fabric was used as a dielectric barrier between the steel and the carbon fiber. Both the carbon fiber and glass fabrics were saturated in a two-part 100% solids epoxy using a mechanical saturator (See Figure 6).

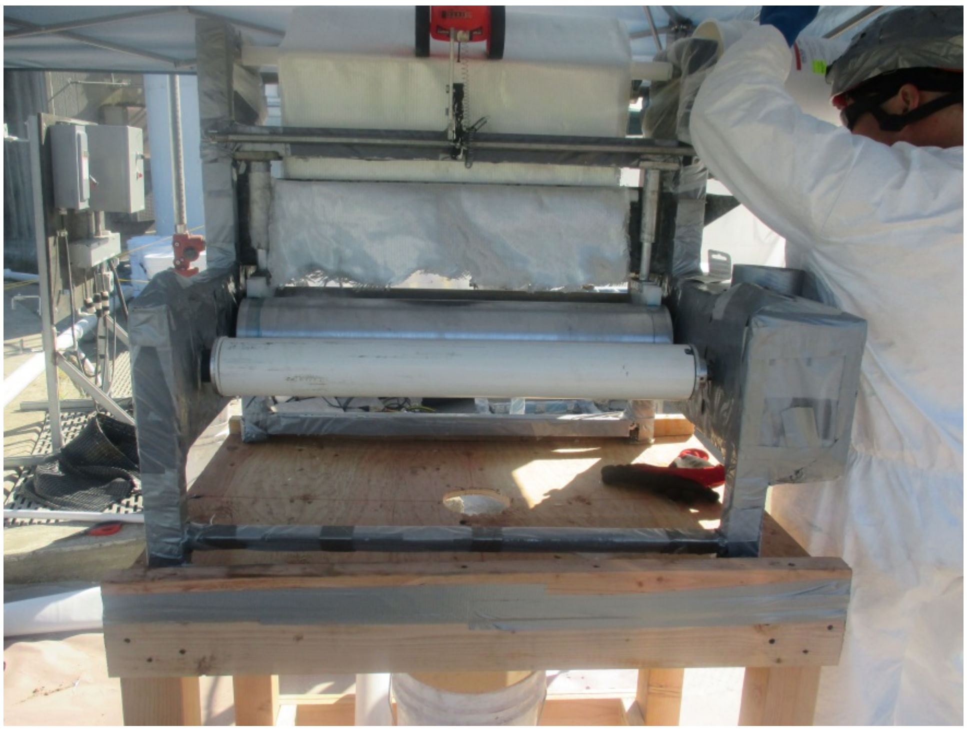

Figure 6. Glass fabric being saturated with a two-part 100% solid epoxy using a mechanical saturator

Prior to application of the CFRP repair system, the concrete substrate was prepared to a minimum of ICRI CSP-3 using sponge blasting per project drawings. In addition, the punctured pipe segments needed to be restored to allow for a uniform substrate during the repair (see Figure 7A-D). Therefore, the inner core and cement mortar in the region surrounding the punctures were removed, as well as the damaged reinforced mortar on the steel pipe section. Any length of soil nails which protruded into the pipe was also removed. Steel plates were then welded over the holes and prepared to SSPC SP-10 near white metal finish via sandblasting. Chemical grouting was used before and after welding of steel plates to address leakage and restore disrupted soil surrounding the pipe segments as needed. The concrete inner core that had previously been removed was then restored using non-shrink cementitious repair mortar, creating a uniform substrate for installation of the CFRP system.

Figure 7A- Puncture in pipeline, as found

Figure 7B- Welding over holes in the pipe

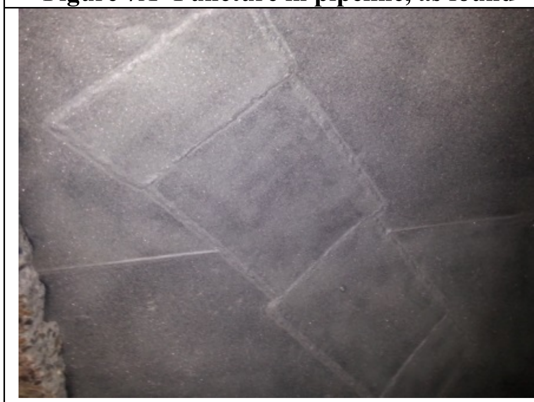

Figure 7C- Prepared substrate

Figure 7D- Surface restored with epoxy mortar

Figure 7. A-D: Restoring punctured pipeline segments to allow for a uniform substrate

In addition to the "typical" dewatering, localized dewatering efforts were required to manage water ingress at the hole locations.

To verify surface preparation, adhesion tests were performed on adjacent substrate per ASTM D4541 to 300 psi minimum (See Figure 8). Once surface preparation was completed, the pipe substrate was covered by a prime coat of epoxy and a layer of thickened epoxy (See Figure 9.)

Figure 8. Pull test per ASTM D4541 being performed to verify surface preparation

Figure 9. Primer epoxy being applied to concrete substrate adjacent to hole repair

Saturated layers of CFRP were then installed onto the interior of the pipe per the project drawings. The design prescribed that the majority of the reinforcement be in the hoop direction and minimal longitudinal reinforcement in straight regions. Additional longitudinal reinforcement was provided near the two (2) 45° bends, see Figure 10 for completed installation.

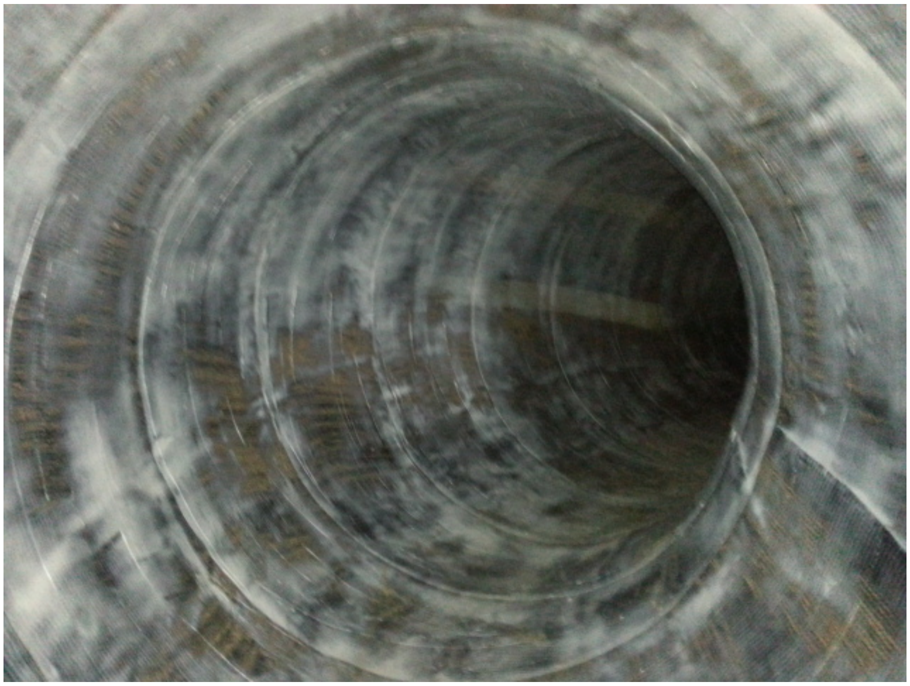

Figure 10. Completed CFRP installation at 45° bend

CONCLUSION

Given the set of circumstances presented for this damaged 54-inch PCCP and the owner requirements to maintain water delivery through peak season, CFRP was applicable and advantageous for the repair. The options analysis and site visit provided the engineer and owner the opportunity to weigh out and thoroughly investigate CFRP repairs of pressure pipelines and the typical construction process. The use of CFRP allowed for a rapid, fully structural repair placing the pipeline back in service with minimal disruption.

REFERENCES

ASTM D4541, Standard test method for pull-off strength of coatings using portable adhesion: *American Standard for Testing and Materials (ASTM)*

ASTM D3039, Standard test method for tensile properties of polymer matrix composite materials. *American Standard for Testing and Materials (ASTM)*

ICRI Guideline No. 310.2, Selecting and specifying concrete surface preparation for sealers, coatings, and polymer overlays. *International Concrete Repair Institute (ICRI)*

SSPC-SP No.10 / NACE 2 Near-White Blast Cleaning. *Society for Protective Coatings (SSPC) and the National Association of Corrosion Engineers International (NACE)*

Repairing the World's Largest Prestressed Concrete Pipe: A Case Study of the Central Arizona Project's Centennial Wash Siphon

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Abstract

The Central Arizona Project (CAP) operates and maintains three (3) 21-foot diameter pre-stressed concrete non-cylinder pipes (PCP) as part of the aqueduct system that delivers almost 1,950 million gallons of Colorado River water per day, to Maricopa, Pinal, and Pima Counties in central and southern Arizona. The pipelines, installed in the late 1970's, have experienced pre-stressing wire breaks and have been repaired several times since the early 1990's. The wires have broken primarily due to defects in the wire. One of the pipes, the Centennial Wash Siphon, conveys water beneath Interstate-10 and the Centennial Wash, about 80 miles west of Phoenix, AZ. The siphon is upstream of CAP's first customer turnout, and as such is critical in the aqueduct's conveyance and delivery system. An internal electromagnetic inspection in January 2013 discovered several pieces of the pipe comprising the siphon had many broken prestressing wires, prompting a closer inspection and assessment of the siphon. The case study described herein examines inspections, assessment, monitoring, and subsequent repair using post-tension tendons of the Centennial Wash Siphon. This paper further discusses the ongoing efforts of monitoring, assessing, repairing, and maintenance practices for the largest prestressed concrete pipes in the world. A brief history of the siphons includes manufacturing of the 252-inch diameter prestressed concrete pipes, installation of the pipelines, and early investigations and repairs. The focus of this paper is on the assessment and monitoring since the last repairs in 2006, specifically newer technologies that have emerged to assist CAP in monitoring and making decisions in the repair methods and locations. Relevant points include a discussion on the excavation of the pipes requiring repair, the repair work (post-tension tendons), and maintenance practices. A brief discussion will follow on new technology recently installed in all three of CAP's prestressed concrete siphons.

Central Arizona Project Background

The Central Arizona Project is a 336 mi (541 km) aqueduct system designed and funded by the Bureau of Reclamation (Reclamation) and operated and maintained by the Central Arizona Water Conservation District (CAWCD) to deliver water to multiple entities throughout central and southern Arizona. In general, the aqueduct system withdraws water from the Colorado River at Lake Havasu in western Arizona and through multiple pumping plants and a combination of open channel canals and closed conduit pipes delivers water to the major urban areas within Maricopa, Pinal, and Pima counties in Arizona – the Phoenix and Tucson metropolitan areas. Water is also delivered to agricultural users, recharge facilities, and Native American reservations along the route. The system is generally comprised of:

- 14 pumping plants
  - Capacities ranging from 208 cfs to 3600 cfs
  - Lifts ranging from 79 ft to 850 ft
- 39 check structures (for water level control)
- 49 turnouts (for customer delivery)
- 6 groundwater recharge facilities
- Two reservoirs
- One Pumping/Generating Plant
- About 350 miles of canals, pipes, and tunnels

Approximately 5 million people (about 80 percent of the population of the state of Arizona) are provided water from the CAP. In 2014, the CAP delivered more than 1.57 million acre-feet (Ac-Ft) of water to various users within Arizona. Table 1 indicates approximate water usage by category.

Table 1. Water Usage by Customer Type

	Acre-Feet(1)	Percent of Total
Municipal & Industrial	440,000	28%
Recharge	251,000	16%
Agricultural	393,000	25%
Native American Reservations	487,000	31%
(1) Rounded to nearest 1,000	1,571,000	Total for 2014

Centennial Wash Siphon History

There are 252 pieces of pipe comprising the Centennial Wash Siphon. Each piece is about 24.5-feet in outside diameter, 22-feet long, and weighs about 225-tons; the siphon is approximately 5,800 feet long. Due to the size of the pipe pieces, onsite manufacturing plants were established near each siphon location and specific manufacturing and transportation equipment was designed for the manufacturing and installation of the pipe pieces. Figure 1 shows one of the field manufacturing plants. Figure 2 shows a piece of pipe being maneuvered into place.

A 1984 failure of a prestressed concrete cylinder pipe on another Reclamation aqueduct system prompted Reclamation to undertake extensive investigations to determine the failure and evaluate future pipeline serviceability. Defective (longitudinally cracked) wire was identified as the cause for the failure of the pipe. Because of the problems Reclamation discovered, they initiated a program to monitor other prestressed concrete pipelines. In January 1990 surveys indicated potential corrosion issues at the CAP's prestressed concrete siphons (in 1990 CAP had six prestressed concrete siphons in service). Widespread distress was confirmed at several locations on all six siphons, and of 223 individual pipe pieces excavated to springline, 28% were found to be distressed and requiring repair, some were so severely distressed complete replacement of the prestressing wire was required. In the mid-1990's Reclamation abandoned three of the six (6) prestressed concrete siphons and installed new siphons paralleling those abandoned. The three (3) remaining prestressed concrete siphons are still in operation – the Centennial Wash Siphon is one of those.

Figure 1 – 1978-79 Onsite Manufacturing Facility

The Centennial Wash Siphon was manufactured and installed in 1978-1979 – the largest prestressed concrete pipe in the world. Results of Reclamation's 1990 investigations resulting from the 1984 pipeline failure lead to the repair of two pieces of pipe on the siphon using post-tension tendons in 1991 and lining the first 1,000 feet of the siphon with an internal steel lining in 1996 – that section of pipeline travels beneath Interstate-10 in western Arizona. Early investigations and attempts to locate distressed pipes were largely unsuccessful; it was not until the early 2000's when the Remote Field Eddy Current Transformer Coupling (RFECTC) Technology was further refined that results became more accurate and reliable, were distressed pipes able to be more accurately identified and located.

In November 2002 the first RFECTC inspection was performed on the Centennial Wash Siphon. The results indicated there were 46 pieces of distressed pipe with the number of broken wires ranging from 20 to 345; just over 18% of the pieces exhibited some level of distress.

Figure 2 – Pipe Mobile (Note Volkswagen for scale)

A second RFECTC inspection occurred in October 2004; the results indicating there were 48 pieces of pipe with the number of broken wires ranging from 10 to 380; just over 19% of the pieces exhibited some level of distress, two new pipes exhibited signs of distress since 2002, and some pipes had an increase in the number of wire breaks. Based on these results it was decided to repair several pieces of pipe.

In November 2006 a third RFETC inspection occurred, as well as repairing several pieces of pipe using external post-tensioned tendons. Results from the 2006 inspection found only one newly distressed pipe, though ten pipes had an increase in the number of wire breaks, and the remainder unchanged from the 2004 inspection. Additionally, an acoustic hydrophone monitoring system was installed to be able to monitor wire breaks in real time. The monitoring system had marginal success; communications to the remote site was problematic and at least once the system broke free of its tether in the siphon inlet due to the high turbulence. By 2010 the system was inoperable so much of the time CAWCD was not able to track wire breaks reliably, so the decision was made to dewater the siphon and conduct another round of Electromagnetic (EM) inspections in January 2013.

In January 2013 a fourth EM inspection occurred and during that outage CAWCD installed an Acoustic Fiber Optics (AFO) Monitoring System to track wire breaks. A new and more reliable fiber optics system for communications had recently been installed along the entire CAP system. The results from the 2013 EM inspection provided a new baseline of distress and provided an idea of wire distress growth since the first inspection in 2002.

The results of the January 2013 inspection, along with the real-time results of the AFO monitoring system for the first few months of 2013 indicated the need to repair several pieces of pipe. The EM inspection indicated 40 pipe pieces had distress ranging from 25 wire breaks to 280 wire breaks, but it was the activity monitored by the AFO system on specific pipes that lead to the decision to conduct repairs. Figure 3 is a graph showing the seven (7) most distressed pieces of pipe based on the January 2013 EM inspection; the graph shows the growth rate of distress since the first inspection in 2002. Also shown are the AFO wire break events for each piece; those numbers are in addition to the numbers reported by the EM inspection.

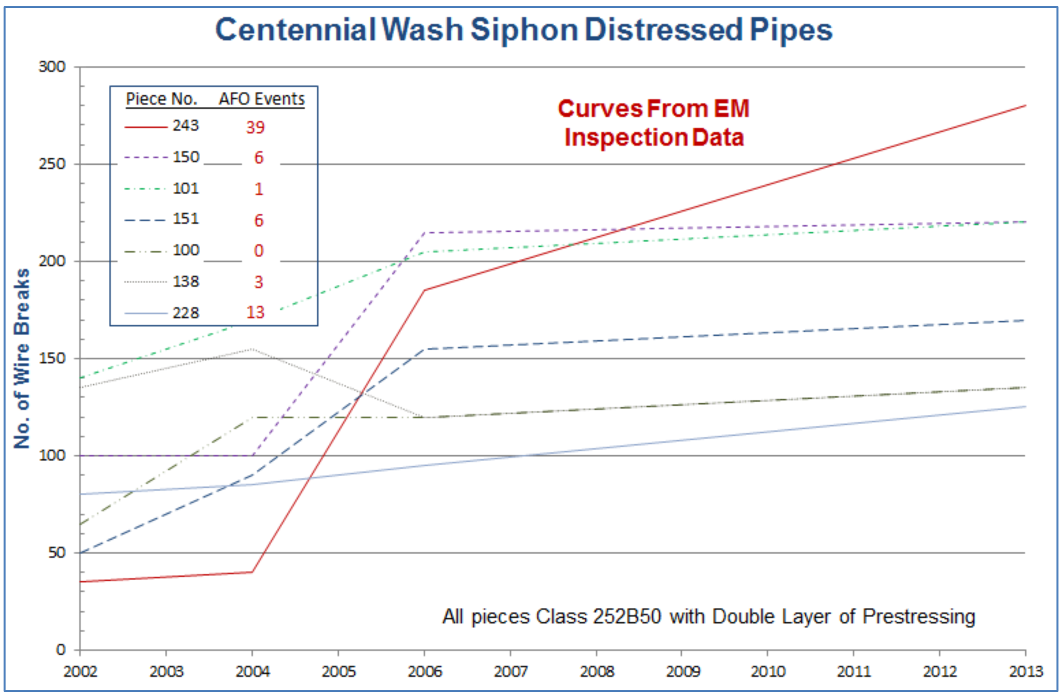

Figure 3 – Graph showing distress growth rate for the 7 most distressed pipe pieces

It takes CAWCD approximately two years to schedule and execute a siphon outage, thus the approximate two years between early inspections. An outage was already scheduled for two weeks in January 2014 for the aqueduct system in the vicinity of the Centennial Wash Siphon, so the decision was made by the Engineering, Maintenance, and Operations groups to fast-track a repair design to be able to hit the January 2014 outage. The design required the repairs be executed within a 12-day period – of the 14-day scheduled outage a day was needed to dewater and another to rewater.

Events Leading to January 2014 Repair

Superimposing the current results of the AFO monitoring system over the EM inspection results from January 2013 and the rate of wire break increase since 2002 enabled CAWCD engineers to determine which pipes should be further investigated to initiate repair. Six pipes were identified and in October 2013 excavated to springline. CAWCD engineers performed acoustic and visual inspections of the exposed pipe pieces and Pure Technologies performed a surface electromagnetic scan to validate the previous inspection data as well as the visual inspections. During the field inspections of the exposed pipes, three additional pipes were identified as requiring repair.

The results of the internal EM inspections, the AFO monitoring system, the surface EM scan, and the visual/acoustic inspections all correlated well, and CAWCD engineers began designing the repair system for nine pieces of pipe. Table 2 provides a chronology of events for the repair effort.

The selected method to repair the pipe pieces was with post-tensioned tendons. Essentially a high strength steel tendon was wrapped around the pipe, tensioned to a specified stress and locked into an anchor block set on the top of the pipe. This required fully excavating portions of the siphon and removing the soil support beneath. Nine (9) pieces of pipe were identified that required repair, but because of their locations along the pipeline, five (5) separate excavations were required.

Excavation of the pipe was conducted in four (4) phases. The first phase (preliminary excavation) occurred in October 2013 when the pipe was exposed to its springline. The second phase occurred in early January 2014 when an additional five feet of material was removed – the siphon was still conveying water for customer deliveries at this point. The third phase removed an additional 3½ feet of material – this took the excavation to the 90° bedding angle of the pipe – at this point water was not being transported through the siphon. The fourth and final phase removed the supporting material beneath the pipes in specific sequences in specified locations, but only after the siphon was isolated from the canal.

The preliminary excavation plan was developed to accommodate quickly and efficiently removing a minimal amount of material during final excavation. It was determined based on the soil composition the excavation would be stable with 1½:1 (horizontal to vertical) side slopes and 3:1 slopes at the ends for vehicular and personnel access. The depth of cover over the pipe varied from just under ten feet to about twelve feet, so excavations to springline averaged over twenty-two (22) feet. If the excavations were cut directly to the springline, the excavations would have been about 80 feet wide at the top; but the intent was to remove as much material as possible during preliminary excavation so a minimal amount was moved during repair. The excavations were cut as if the side slopes proceeded to a point one-foot below the bottom of the pipe which set the width of the excavations at about 130-feet. The 3:1 end slopes dictated an excavation over 200 feet longer than the length of exposed pipes. Excavation accounted for about 80% of the construction duration.

Table 2. Chronology of Events Leading to Repair Effort

Time Frame	Activities
January 2013	EM Inspection
Feb-Apr	Tracked AFO Events
Mar-Apr	Evaluated EM Inspection Results
May	Established Repair Priority
May-Jun	Prepared Internal Planning Documents and Preliminary Design for Project Approval
July	Received Senior Management Approval for Project
Jul-Sep	Developed Preliminary Excavation Plans
October	Preliminary Excavation – to Springline
November	Surface EM Scan – Validation of Inspection Results
Oct-Dec	Visual Inspection of Exposed Pipes Development of Repair Contract Documents Selection of Repair Method – Post-Tensioned Tendons
6 Jan 2014	Began Additional Excavation
13 Jan 2014	Began Outage – Contractor Began Repairs
25 Jan 2014	Contractor Substantially Complete
26 Jan 2014	Siphon Put Back in Service

Current Repair (January 2014)

In mid-January the water was "turned off" to the siphon and the siphon isolated from the upstream and downstream canal sections. Stoplogs were set at the siphon inlet and outlet and a pump positioned in the siphon to begin dewatering the siphon – the intent was to remove the water pressure in the pipe. At this point the Contractor was allowed to remove material from around the pipe to the 90° bedding angle (third excavation phase).

Excavation beyond the 90° bedding angle (to beneath the pipe) was only allowed in the locations where the pipe was to be wrapped with post-tensioned tendons. The sequence of repair limited excavation beneath the pipe to 12-feet at mid-span and 10-feet across a joint; this required three separate excavations to wrap a single 22-foot piece of pipe.

During design, a repair sequence was established that optimized the sequence of repair areas. The repair areas would be excavated, wrapped with tendons, the tendons stressed, and the pipe backfilled with a controlled-low-strength-material (CLSM) up to the 90° bedding angle, see Figure 4. The CLSM is essentially a "one-sack slurry" (flowable fill) to mimic the original bedding of the pipe and provide support to the pipe haunches that was removed to facilitate the repair. The CLSM also provides a higher pH buffer around the pipe and new tendons from the soil. Once the CLSM sufficiently cured, the material adjacent to the CLSM could be removed and repairs undertaken in the new area. There were 17 separate areas for which the following repair sequence was performed (at this point the pipe was only "buried" up to a point of about 3½-feet of soil):

- Excavate to one-foot below the pipe
- Wrap post-tension tendons around the pipe
- Stress the tendons and lock them in the surface couplers (anchors)
- Backfill the repair area with CLSM up to the pipe's 90° bedding angle – ensuring complete flow beneath the pipe

Figure 4 - Placing CLSM Beneath Pipe

Figure 5 shows two pieces of pipe excavated where the tendons have been installed, some of which have been tensioned. Note the material removed from the mid-span of adjacent pieces and the pipe is supported under the joints.

Figure 5 - Sequencing of Repairs

When the post-tensioned tendons were installed on all nine (9) pipes requiring repair and all areas backfilled with CLSM up to the 90° bedding angle, CAWCD was able to begin conveying water through the siphon – Substantial Completion was given at this point – two (2) days prior to the end of the outage. The next task was to bond the anchor blocks to each other and to sacrificial anodes for eventual connection to a cathodic protection test station. Each pipe that was repaired had two anodes and a test station installed. The next sequence was to apply a three-inch thick layer of shotcrete over all the repair areas to completely encapsulate the tendons and anchor blocks in a high pH environment to minimize corrosion of the anchor blocks and add additional protection to the pipe.

Once the shotcrete cured the process of backfilling the excavations began. Material was replaced in the excavation in 12-inch lifts and compacted up to 5-feet below the springline of the pipe. From 5-feet below springline up to final grade material was placed in 18-inch lifts and wheel rolled – no special compaction was required. Cathodic protection test stations were installed and tested upon completion of backfilling.

Post-Tensioned Tendon System

The post-tension tendon system selected was from Dywidag-Systems International (DSI) which is generally composed of a high strength tendon (ASTM A416), an anchor block that was installed on the surface of the pipe, and a three-part wedge that locked the tendon into the anchor block. The typical sequence of operation was the Contractor wrapped several tendons around the pipe, typically at a 6-inch spacing, set each tendon in an anchor, tensioned the tendon to approximately 47,000 pounds, (~216,000 psi), and locked the tendon in the anchor. The tendon relaxed to about 41,000 pounds, ~189,000 psi when locked in the anchor. The anchors were then injected with a rust inhibiting grease and bonded to each other prior to being covered in a protective layer of shotcrete.

Continuing Assessment and Monitoring

CAWCD is actively monitoring all three prestressed concrete siphons with the recently installed Acoustic Fiber Optics Monitoring Systems and is constantly reviewing results with the current and past EM inspection results to look for trends in individual pipes pieces, as well as gauge the overall health of each pipeline. Failure curves for each pipe class were recently developed and those curves are being used to assess the pipes and even trying to predict future events based on trending.

CAWCD is also in the process of developing a "Pipeline Reliability Group" to actively meet and discuss the condition of all of CAP's pipelines, not just the siphons. The group, composed of engineering, operations, and maintenance personnel will make recommendations on further monitoring activities, as well as schedule and begin planning for additional inspections. CAWCD has experience in monitoring, assessing, and maintaining pipelines, and continues with that experience in a proactive manner. If repairs are required, it is this group that will support the engineering team in developing and executing repairs.

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Motts Run Dam Outlet Rehabilitation—A Case Study Illustrating Design and Construction Aspects

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Abstract

This paper presents a case study covering design and construction phases of sliplining the outlet pipe of the Motts Run Dam in Spotsylvania County, Virginia. The reinforced concrete outlet pipe had been previously lined with an HDPE liner in order to allow the outlet pipe to be used for pumping water back into the reservoir. However, the liner did not perform as expected. An investigation of the liner determined that the existing HDPE liner had experienced major deformation. The County decided to remove the failed HDPE liner and replace it with a steel liner. The paper provides an overview of the challenges encountered during design and construction. The project was successfully completed in 2015.

INTRODUCTION

Background and history. Motts Run Dam is located about eight (8) miles upstream and west of Fredericksburg in Spotsylvania County, Virginia. The dam is located on Mine Run tributary, approximately 1,000 feet upstream of its confluence with Rappahannock River. With a storage volume of over 4,000 acre-feet, the Motts Run Reservoir is an important source of raw water for the Spotsylvania County's Motts Run Water Treatment Plant. The reservoir consists of an earth fill embankment dam, constructed in 1970's, with a separate intake tower that also serves as the principal spillway for the dam. The 700-ft long, 48-inch RCP outlet pipe extends through the dam embankment and connects the intake tower to the outlet structure. The outlet pipe is designed to convey overflow and intentional releases from the reservoir under gravity.

In 2002, during a previous improvement the outlet pipe was lined with a 36-inch HDPE liner and the annular space was filled with cellular grout, with an intention to utilize the outlet pipe for filling the reservoir from Rappahannock River Intake pumping station. These improvements were intended to utilize the reservoir as pumped storage which required the pipe to operate under pressure. In 2008, the County needed to raise the water level in the reservoir and routed the pressurized flow from the Rappahannock River Intake Pumping Station through the outlet. While the pipe was pressurized, a concentrated seep was observed on the downstream slope of the dam which threatened the integrity of the dam. The pipe was depressurized and an investigation was performed to identify the cause of the seepage. A CCTV inspection of the outlet pipe was performed in September 2008. The inspection revealed significant lengths of the HDPE liner had undergone minor to moderate distortion. The inspection also revealed several shorter lengths with severe distortion. Due to concerns associated with the safety of the embankment, a Ground Penetrating (GPR) survey was performed from inside the outlet pipe to assess the potential existence of voids behind the HDPE liner and the RCP pipe. Three areas with large voids along with several minor voids were identified. Figure 3 shows the summary of the GPR results. However, due to the signal strength it could not be confirmed if the voids were in the embankment or in the annular space.

To further assess the condition of the embankment, a Cone Penetrometer Testing (CPT) program was conducted on the downstream slope of the embankment. The objective of the cone testing was to identify embankment distress resulting from seepage through the downstream slope. Areas near the concentrated seep locations were investigated for underlying soft or loose material that would indicate weak zones in the embankment. The CPT investigations did not indicate presence of any unusual weak zones that could potentially pose any risk to the dam. Based on the results of the aforesaid investigation, it was concluded that the HDPE liner deformation was likely due to uncontrolled grouting operation of the annular space during the previous modifications. It was concluded that remedial measures were needed to allow the County to use the outlet under pressurized conditions.

Purpose and scope of current rehabilitation. The purpose of the current rehabilitation is to make the outlet pipe capable of conveying pressurized flow from the pump station into the reservoir. The County identified other improvements for enhancing operational efficiency and reliability of the reservoir. These improvements included works in the intake tower and at the impact basin. The following is the description of the work scoped under the current rehabilitation:

Works at the outlet pipe. The works included reaming out of the existing 36-inch HDPE liner from the outlet pipe, and re-line the outlet pipe with a new 36-inch steel pipe. Upsizing of the liner was considered but was deemed unnecessary from a capacity standpoint. Additionally, there was a concern that the original host pipe could have joint misalignments or bellies which could interfere with the installation of steel liner. As such, the size of the liner was keep at 36 inches. The annular space between the new steel pipe and the existing 48-inch RCP was filled with cement grout. Since the GPR results exhibited potential voids in the embankment outside the existing 48-inch liner, another GPR survey was included in the scope after removal of the existing HDPE liner and annular grout. The replacement of the existing liner entailed demolition of the existing impact basin, the outlet gate valve and ancillary pedestrian bridge structure. The new design included replacement of the gate valve and modification in the ancillary structures to enhance flexibility in operation and maintenance of the outlet valve.

Miscellaneous works at the intake tower. To take full advantage of this opportunity, other miscellaneous works identified separately were included in the scope of this rehabilitation. These works included replacement of existing sluice gates and installation of a new low level outlet. The existing sluice gates exhibited displacement, causing leaks and difficulty in operations. In order to mitigate future potential for leakage, the new gates were relocated on the outside face of the intake tower walls, and were installed using through-bolt connections. In addition, a new low level outlet valve was installed in the intake tower as a replacement for the existing 36 inch low level outlet that was not operational.

Challenges during design and construction. The design and construction of these works were challenged because of limited information from previous modifications, unknown structural conditions of the 48 inch outlet pipe, and limited ability to assess conditions of the embankment from within the HDPE liner. Additional planning for flood mitigation and worker safety was required since working within the outlet pipe would potentially require closure of the principal spillway. The County worked with the Engineer to clearly define project objectives, identify elements that are critical for long term performance, and develop Contract Documents to accomplish the desired level of quality control during construction. The Contract Specifications included specific measures such as CCTV inspection of host pipe at several stages of liner demolition, quality control of welds for the new steel liner, extensive review of liner placement procedures in the outlet, low shrinkage cementations grout, contact grouting, detailing of end connections to block potential seepage paths and pressure testing. Extensive coordination between the Engineer and the Contractor was enforced to ensure continuous evaluation of Contractor's means and methods with the progress of the project and with the changes in condition that were not anticipated during the design. The following sections present aspects of design and construction that were critical to the successful completion of the project.

CRITICAL FACTORS - DESIGN AND CONSTRUCTION PHASE

The most critical aspect of the design was to put together Contract Documents that clearly define the project objectives while outlining constraints and risks for the Contractor. Since the dam is a critical part of County's water supply infrastructure, several opinions from the Engineer's in-house technical experts and outside contractors were sought to review constructability, identify anticipated means and methods, potential risks during construction and feasibility of the design. The process included in-depth discussions with the County staff to lay out available options, jointly evaluate potential risk and rewards for these options, and choose the best course for the project. This section summarizes critical factors encountered during design and construction, and the mechanisms adopted to address those factors.

Demolition of the existing HDPE liner and annular grout. The demolition of the existing 36 inch liner and annular grout was identified as the critical task with highest degree of risk and uncertainty. There were limited as-built records from 2002 modifications that forced the designers to base the current design on previous design drawings. Elements such as the strength of annular grout used in construction added uncertainty associated with the hardness and associated difficulty in its removal. There were concerns that if richer mixes were used to exceed the specified strength parameters in the previous modification, it may make it difficult to demolish the grout without using mechanical means. There was a concern that the host pipe could be damaged during demolition of the liner and annular grout

Similarly, several possibilities were contemplated pertinent to the deformed condition of the existing 36 inch liner. Based on preliminary assessment, the deformation was likely due to uncontrolled grouting operation of the annular space during the previous modifications; however, the indication of potential voids in the GPR survey raised concerns that the host pipe (i.e. existing 48 inch RCP) might have cracked leading to loss of material along the concentrated seepage path, and subsequent washout of embankment material into the outlet.

To mitigate these concerns, specific measures such as CCTV inspection of host pipe at several stages of liner demolition were included in the Contract Specifications. The Specification required the Contractor to perform CCTV before and after removal of the annular grout to preempt any risk resulting due to the damage of the host pipe. The Contractor was limited to demolishing the liner and grout in 75-feet sections to reduce the risk of any unforeseen circumstances resulting from the demolition. The Contractor could advance the demolition only after satisfactory review of the CCTV data by the Engineer for the previous 75 feet section. Enforcing these provisions of the specifications required a proactive approach, and defined protocols to efficiently minimize impacts to the construction schedule. The Engineer worked closely with the Contractor to coordinate the timing of the CCTV review and provide resources for expeditious reviews and decisions. The Contractor used a small hand held equipment to cut the HDPE pipe into small pieces and to demolish the annular grout. It was found that the bond between the annular grout and the host pipe was very weak and big chunks of grout would come off when impacted by a hammer and chisel. The outlet pipe was found to be in good condition. The results of a GPR performed after the removal of the liner and grout did not indicate the presence of any voids. As a result, minimal repairs and grouting of the areas outside the outlet pipe was required. Figure 1 shows the view into the host pipe following a section of HDPE and annular grout were removed.

Figure 1. View inside 48 inch RCP after removal of HDPE liner and annular grout

Temporary diversion. The rehabilitation of the outlet pipe required accessing the pipe from the downstream impact structure, which necessarily required closure of the principal spillway. Although, there is an adjoining emergency spillway which is sufficient for the safety of the dam embankment, the configuration of the reservoir and emergency spillway does not permit routing of spills or small discharges through the emergency spillway. The two options were either to provide storm storage within the lake by lowering the reservoir, or to design a temporary by-pass mechanism that could divert inflows from the lake to the Rappahannock River.

Lowering of the lake by a few feet was deemed necessary for the safety of the workers, considering required reaction time to vacate the outlet pipe in the event of an unprecedented spill from the lake. However, since the reservoir accounts for a third of County's raw water storage, the County was hesitant to lower the lake in view of potential water supply concerns during the peak summer demands. Since the lake serves as a key recreational facility for the residents of the City of Fredericksburg, there were additional concerns that the lowering of lake would negatively impact recreational activities along the periphery of the lake. During the design phase, these aspects were discussed with the County to determine a workable approach without shifting too much risk on the Contractor.

It was decided to adopt a middle of the road approach that included lowering of the lake level by a few feet and including a temporary diversion to suit Contractor means and methods. The approach essentially provided flexibility to the Contractor to size the diversion while offering a maximum limit to the allowable lowering of the lake levels. The maximum allowable lowering was set at 4 feet that could provide temporary storm storage for a 5-year storm event. In addition, optional temporary raising of the weir elevation of the principal spillway by 2 feet (from the existing normal pool) would allow a combined storm storage equivalent to a 10-year storm. The Contractor utilized this approach effectively by using a multi-pipe siphon by-pass system to siphon the desired amount of water from the lake. Once primed by small pumps, the siphons provided flow under gravity on a continuous basis. The number of pipes was adjusted during the course of the construction, depending on the amount of inflow into the lake. During construction several storm events were successfully contained within the reservoir, and gave enough time to the Contractor to vacate the outlet, remove equipment and prepare the site for potential spills through the principal spillway. The specified method of temporary diversion was successful, and proved to be a cost-effective solution for the project. Figure 2 shows multiple siphon pipes used as temporary diversion.

Figure 2. Downstream view from the top of the dam showing siphon pipes

Encourage innovation from the contractor. During design, the engineering team reviewed the anticipated means and methods that the Contractor could potentially use for the project. Because of the unconventional nature of the work, it was envisioned that subject to review and approval, the design may accommodate innovative means and methods from the Contractor to provide flexibility. This philosophy was engrained in the design drawings and specifications that contained a representation of a basic work scheme that would ensure constructability using a relatively low risk approach. The work scheme was deliberately made conservative to invoke Contractor's interest in the opportunity and motivate him to innovate for financial gain.

The approach not only invited fresh thoughts and innovation from the Contractor, but also protected County's interest since it guaranteed that the Contractor would deliver the work product as specified. One drawback of the approach was that it required more review and coordination during construction; however, given the unconventional nature of the work and the lack of previous engineering records, the overall benefits of the approach outweighed the additional engineering time during construction. An example of this approach was to keep provisions for alternative methods of installations for the steel liner.

The design for the installation of the steel liner was based on the assumption that the steel liner would be assembled by butt-welding pipe segments over a temporary platform on the downstream side of the outlet. Following the assembly, the liner would be pushed using conventional hydraulic jacks into the host pipe. The design specified the use of flexible spacers to prevent the liner from floatation during grouting of the annular space. The cementitious grout specified for filling the annular space was designed to act as a corrosion shield on the exterior of the steel pipe. It was envisioned that this option minimizes extensive welding within the host pipe and hence most workable. However, the Contractor submitted an alternative method that was based on assembly of the liner inside the host pipe. The method did not require pushing the assembled liner, rather it utilized adjustable jacking bolts to centralize the pipe. Individual pipe segments were to be butt-welded from inside by using a steel backing plate at the joints. Following the grouting of annular space, the jacking bolts would be unscrewed and capped using steel plugs.

The primary advantage of the method submitted by the Contractor was that it allowed adjustments in the liner on account of unforeseen bends or kinks in the host pipe. The method also provided ability to align each segment individually by tightening or loosening the jacking bolts. However, there were concerns pertinent to structural integrity of the pipe due to concentration of stresses at jacking locations during grouting. Since the Contractor had planned to perform annular grouting in a single stage, the jacking locations at the top of the liner were potentially the most affected due to high stresses resulting from floatation. Additionally, the method submitted by the Contractor required annular grouting to be performed in sections using temporary bulkheads. There were concerns that the shrinkage of the grout mass may allow infiltration that could reach the surface of the steel pipe through these joints that could potentially corrode the steel pipe.

During the review of Contractor's submittal, the engineering team performed a stress analysis using a 3-D model that concluded that the stress concentration on the top jacking bolts location was about 25% higher than the allowable limits. It was recommended to strengthening the pipe at these locations using an added plate. In addition, secondary grouting was recommended to mitigate concerns due to shrinkage and prevent any compromise in the corrosion protection of the steel pipe. The Contractor agreed to modify the design and the installation was successfully completed. Following the grouting, the liner was successfully inspected and pressure tested.

Figure 3. Completed outlet structure with new gate valve and removable thrust blocks

CONCLUSIONS

The project was completed successfully with minimal design changes during construction. The total value of the Change Orders paid to the Contractor on account of these design changes was less than 5% of the total Contract Value, and well within the contingency. Barring delays due to the bad weather, the project finished on schedule and without any delays on account of changed conditions during construction. This is particularly important since the Contractor had to work with unknown conditions, and plan for contingencies ahead of the time to be on schedule. The project was a well-managed team effort where the County actively participated with the Engineer and the Contractor to pre-empt issues, risks and take timely decisions to keep the project on track.

Among several factors that contributed to the success of the project, the key factors were preparation of sound Contract Documents, identification and management of risk, and extensive documentation during construction. This required extensive engagement between the County and the Engineer during the design phase to discuss potential options, risks and outcomes during construction. The engagement was extended during construction to include the Contractor to be able to discuss the project issues, and bring fresh thoughts on board to steer the project in the most beneficial manner. The project execution not only catered to the current project needs but also secured information for future by documenting critical project communications, decisions during construction and field observations that will serve as reliable records for future.

Design and Construction of a Raw River Water Welded Steel Transmission Main for a New Water Supply System in Northern Virginia

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Abstract

Construction is ongoing of a 100-year service life 5.25-mile 42- and 48-inch welded steel pipeline, conveying raw river water from the Potomac River for a 40 MGD water supply system in an urbanizing county in northern Virginia. Ductile iron pipe (DIP) and welded steel pipe (WSP) were specified to create price competition. Transient modeling incorporated material-specific celerity values and showed that vacuum due to surge was more severe than positive surge. Air vacuum relief valves were located at each high point and 4 non-high points where transient surges were predicted. Cathodic protection was designed, consisting of a bonded coating and galvanic anodes. All bids received were for WSP. Lay schedule and shop drawings review assessed alignment, joint types and pulls. Constrained easements prompted reduced radii elbows. The heat shrink sleeve dissipates heat from joint welding after backfill. Daily lay production of 250 feet is planned. The strength of single lap welds was considered. Each weld is to be magnetic particle tested.

Evaluation of Six Materials Results in DIP and WSP Specified in Documents

Patterned after pipeline material evaluations often developed for large diameter water transmission mains in the Western U.S., there's a growing trend for Owners and designers in the Eastern U.S. to perform a similarly comprehensive evaluation for projects with a significant first-cost investment, i.e. in the range of 60 to 70 percent of life cycle cost. For the Loudoun County, Virginia raw water transmission (RWT) project, a robust evaluation outlined performance requirements and assessed each material's ability to offer long-term reliability—an uninterrupted 100-year service life—at an acceptable, budgeted cost. Such an evaluation may challenge an Owner's standards by bringing to bear current data and empirical evidence of failure modes, availability, and constructability, for each pipe material. Design of the RWT project included a comparative evaluation considering six pipe materials: ductile iron, welded steel, bar-wrapped concrete, PCCP, HDPE and PVC. Each material was scored under the following criteria, in priority order: i) total installed cost; ii) availability and demonstrated experience in required diameters and pressure classes; iii) life cycle cost; and iv) failure mechanisms and history. An example scoring for life cycle cost considered pump power consumption, which translates to the pipe's ability to remain smooth over its service life. Selection criteria scoring of the six materials proved welded steel pipe (WSP) and ductile iron pipe (DIP) to be finalists for the project, and were accordingly specified in the documents to take advantage of market competition. The two variables that most affect the choice between DIP and WSP, while also being the primary drivers of cost differences, were wall thickness and external coatings. Wall thickness gives the pipe its principal resistance to structural failure while an external coating is the primary means of corrosion protection from surrounding soil and externalities such as stray current. While developing bidding documents, the engineer defined these two variables for each pipe type to ensure Contractor bids are based on two equally performing materials.

Wall Thickness Design: DIP and WSP are manufactured differently. Ductile iron pipe is centrifugally cast with the deLavaud process, while WSP is helically welded from coiled sheets. DIP and WSP have similar ultimate and yield tensile strengths, but different elongation, hinting at a key difference between the two materials: toughness. Generally, WSP's higher elongation translates into greater toughness compared to DIP. In turn, different toughness ranges lead to material specific wall thickness design methods per AWWA, although methods for both consider the material's resistance to internal loads (pressure design) and external loads to arrive at a required wall thickness. Owing to its long segment lengths, WSP wall thickness design also requires a handling check to evaluate its beam strength. The design methodology for internal pressure loading of both materials uses the Barlow Hoop Stress calculation to estimate wall thickness required to resist the maximum expected hoop stress, which is a circumferential tensile stress of greatest value along the pipe's inner diameter. Application of the hoop stress equation to WSP and DIP, per AWWA methods, introduces built-in conservatism such that computed wall thicknesses results in a relatively small likelihood that the minimum yield stress will be developed in the pipe wall due to static or surge pressure. For the RWT project, with surge suppression devices in place—air release and vacuum relief anti-shock values at each alignment high point, as well as four specific non-high point locations, and a surge relief tank on the immediate discharge side of the proposed pump station—the maximum steady state pressures are comfortably less than 200 psi closest to the pump station, and less than 150 psi for the majority of the line; hence, pressure class 150 and 200 DIP is the analytical solution for wall thickness to resist internal loading. Accordingly, the equivalent WSP wall thickness to resist these internal pressures are 0.188 and 0.208 inches for 42- and 48-inch pipe, respectively, based on the following AWWA-prescribed Barlow Hoop Stress calculation procedures for test and working pressures: the allowable tensile stress, i.e. hoop stress, in the pipe wall during working pressure can be up to half the steel's minimum yield strength (0.5*42ksi), while that fraction during test pressure is up to two thirds (0.67*42 ksi). These relatively thin-walled pipe solutions were overridden in the final specifications by the Owner's standard thick-walled pipe: DIP thickness class 52 (0.59 and 0.65 inches for 42" and 48" DIP respectively), which translated to a WSP thickness of 0.3125 and 0.355 inches for 42 and 48 inch pipe respectively, using the aforementioned fractions of minimum yield strength for allowable wall hoop stress during working pressure (0.5*42 ksi) and test pressure (0.67*42 ksi). To underscore a clear difference between current standards of practice in Western and Eastern U.S. geographies—relatively thin-walled transmission mains are common in the west as long as an external bonded coating and a cathodic protection system are specified. However, this standard is not as prevalently accepted for projects in the Eastern U.S.. The cost implications of these thick walled pipe specifications may be highlighted by considering payment for pipe as dollar per pound of metal, versus the conventional dollar per linear foot. Applied to the thin-walled pipe, this payment is dollar per pound of metal required for performance, while payment for thick-walled pipe defined in the project's bid documents is dollar per pound of metal specified.

Two Sets of Vaults Designed**:** The time and temperature controlled annealing operation of the deLavaud process causes DIP to be less suited to welded outlets, as welding heat stresses weaken the metal microstructure. Consequently, structural failures of welded connections and outlets are acknowledged. If welded connections or outlets are used, the manufacturer shall be consulted to discuss limiting the heat affected area of the parent pipe, restraining lateral joints, and minimizing moment arm loads onto the weld. It's noteworthy that one DIP manufacturer reinforces their welded appurtenances, similar to the reinforcing collar and wrapper plates used on WSP welded appurtenances. The represented consulting Engineer has helped a DIP manufacturer develop testing protocols for welded appurtenances to evaluate resistance to moment arm failure, as opposed to the previously-held focus on axial thrust loads. DIP manufacturers often require special thickness class 53 parent pipe when fabricating welded appurtenances.

Savings Predicted if WSP Selected over DIP: Two independent factors governed the direct cost of WSP and DIP for this project: wall thickness and exterior coating. Pertaining to 42-inch pipe, Thickness Class 52 DIP—with 0.59 inch wall—generally is more costly than the equivalent performing WSP wall of 0.3125 inches. As introduced above, the cost implications of additional metal comparatively disadvantaged DIP. This disadvantage is compounded by the cost increase imposed by the specified dielectric bonded coating for DIP versus WSP, as detailed in this paper's cathodic protection section. When pairing wall thickness and exterior coating specifications, WSP was predicted by the Engineer's estimates to be approximately $4M less than DIP (in this case, a 15 to 20 percent savings in first cost for the entire project. This anticipated savings was likely realized as all contractor bids were for a welded steel pipeline. Moreover, a pipe manufacturer that offers both WSP and DIP was likely advantaged by volume pricing when coupling the WSP pipeline project with DIP on the program's water treatment plant project. The adopted approach to create market competition between materials within the bid documents seeks to optimize derived value to the Owner, as manufacturers are not only competing within a material—for example, DIP manufacturer A versus DIP manufacturer B—but also across materials—DIP manufacturer A vs. DIP manufacturer B vs. WSP manufacturer. Ultimately, a pipe vendor who manufacturers both DIP and WSP earned the supply contract with the general contractor.

Alignment Design Based on DIP Deflections, WSP Segment Geometry Customizable

The below figure is the detail shop drawing of a special segment of WSP: a 60 degree bend shop welded on an otherwise straight length of pipe, which sharply contrasts the equivalent 2-piece DIP arrangement: a straight piece jointed to a 60 degree bend fitting with mechanical restraint. The WSP solution eliminates a field joint and mechanical restraint apparatus and hence expedites laying production while reducing leak potential. Other than 90 degree elbows, which are individual pieces, horizontal and vertical bends are generally accomplished in this manner with WSP.

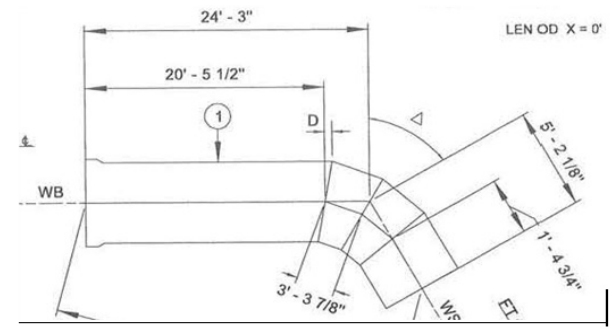

Figure 1: Steel pipe segment for 60 degree bend from American Spiral Weld Pipe Company.

Steel's geometric solution for this highly constrained alignment—both horizontally due to a highly developed landscape and vertically due to crossing utilities, streams, and roadways—relies on mitered bells and field joint pulls. Acknowledging the bid documents originally prohibited field pulls at mitered bells, this restriction was lifted by the Engineer during lay schedule development as requested by the manufacturer. This request stemmed from the project's distinctive need for multiple consecutive mitered joints to accommodate horizontal and vertical curves. Pulling at mitered bells prevented an abundance of unique mitered bell shop cuts, which would have sharply increased the cost of fabrication. While this provision was lifted, the Engineer retained the stipulation of a maximum allowable five degrees per pipe joint of total allowable deflection, which is the sum of miter and field pull angles. The Contractor is especially appreciative of the flexibility for field fit when the Engineer permits field pull. This five degree rule is rooted to this pipeline being a relatively low pressure installation, and hence comparatively low induced thrusts when considering steel's allowable wall stresses. Joints that are both mitered bell and field pulled are well served to also be restrained, i.e. welded, as the wall stress intensification linked to this five degree specification is minimized by the reinforcing afforded by the weld. Furthermore, it's notable that mitering of spigot pipe ends was prohibited.

Transient Modeling of Delivery System

Startup and future operation of the new water supply system's four elements were integrated into a steady state and transient hydraulic model to predict the pressure envelope faced by transmission main piping. The following four flow modes were modeled: 1) river water pumped from river pump station (RPS) to quarry storage; quarry water was then subsequently pumped by quarry pump station (QPS) to water treatment plant (WTP); 2) river water pumped from RPS to a flow split at control valve vault to deliver to both the WTP and quarry storage, with the QPS simultaneously pumping to WTP; 3) river water pumped by RPS strictly to WTP; and 4) river water pumped by RPS to a flow split at control valve to deliver to both quarry storage and WTP. The RPS features 3 duty pumps, each rated at 9,236 gpm at 297 feet TDH, while the QPS features 4 duty pumps, each rated at 6,950 gpm at 430 feet TDH. The target transient pressure envelop was -7 to 250 psi, with -7 based on minimum pressure at a steel or DIP gasketed joint and 250 psi based on an economical steel pipe wall thickness and DIP pressure class pipe; moreover, applied conditions coupled the lowest TDH and highest C factors to yield maximum velocities. Transient modeling input parameters included: i) pump and motor moments of inertia; ii) fastest wavespeed (celerity) between WSP and DIP; iii) response time of pressure reducing valve at the flow split vault and WTP influent valve; and iv) double orifice air-vacuum valves and a surge tank at the RPS and QPS. Pump trips at the RPS and QPS were modeled separately and simultaneously to establish the allowable pressure envelope. A total of 25 different transient scenarios were modeled, resulting from the combinations and permutations of the described flow modes and transient events (*i.e.*, pump starts, pump trips, valve closures, etc.). Air-vacuum valves' location and size were adjusted iteratively until pressures fell within the desired pressure-vacuum envelope. Refinement model runs were then performed with normal pump start/stop in conjunction with valve closure at maximum and emergency rates. Thereafter, "hydraulic capacity" model runs sought the maximum flow the system could deliver pressures while maintaining the established pressure-vacuum envelope. Once aggregated, model results informed the selection of pipe wall thickness, restrained joint lengths, and the size and location of air-vaccum valves and hydropneumatic surge tanks, as presented by bid documents. Conclusions that significantly affected design include the following: 1) abnormally high pressures were comfortably addressed, while mitigating full vacuum conditions proved difficult; 2) the optimal size of air-vacuum valves was 8 inches, which is not the largest offered by the manufacturer (12 inches); 3) slow valve closure rates (in some cases, up to seven minutes for full closure) significantly mitigate transient responses; 4) once future demand triggers the need for larger pumps at the RPS and QPS, their larger moment of inertia should improve the system's transient response; and 5) a surge tank at the RPS and QPS will be relied upon to minimize vacuum conditions, i.e., the most threatening transient event. Several instances of cross-contract coordination were incorporated into this modeling effort, e.g., the RPS surge tank was integrated into the river intake and pump station contract, not the pipeline contract.

Robust Cathodic Protection System

The first step taken during design to determine the need for cathodic protection (CP) was obtaining profiles of soil resistivity measurements along the proposed alignment and comparing the results at the proposed pipe depth with benchmark soil corrosivity values. Widely used sources of benchmark corrosivity data include the Bureau of Reclamation (dominant in western U.S.), NACE/Corrosion Consultants (empirical; dominant in eastern U.S.), and the AWWA Corrosion Control for Buried Water Mains Pocket Field Guide. Disparate resistivity values across these sources that define corrosion severity categories, e.g., most severe soil environment, engendered the need for Engineer judgment. For example, a 6,000 ohm-cm resistivity measurement benchmarked against the Bureau of Reclamation's guidelines suggests DIP and WSP do not need cathodic protection—AWWA C105, Polyethylene Wrap and AWWA M11 would also suggest soil corrosion is of low likelihood; however, the NACE and AWWA Pocket Field Guide classification regimes would indicate a corrosive and moderately corrosive soil environment respectively, justifying a decision to cathodically protect the pipeline. Given the 100-year service life objective, the set of NACE classifications was adopted for this project, and when contrasted against field soil resistivity measurements, nearly 83% of the data demonstrated a corrosive to very corrosive soil environment. When paired with the stray current threat from adjacent and crossing gas mains (which employ active impressed current systems), the need for CP became paramount.

As the bulk of the pipeline progresses within an existing buried and overhead utility corridor, namely natural gas mains and high voltage power lines, the Engineer was alert to stray current and alternating current induction. While soil corrosion threatens long term viability of the transmission main, the predominant rapid corrosion failure hazard emanates from stray current from two nearby and crossing 30-inch high pressure natural gas mains, each configured with an impressed current cathodic protection system. One main's operating rectifier is rated at 47 volts and 5 amps, while the second main features a rectifier on each end of the roughly 6,000 foot length over which it parallels the RWT main—rated at 36 volts and 4 amps, and 45 volts and 3 amps. These large protection currents act as strong stray current sources, which was a primary factor for the bonded coating specification. Should gas main protection current stray onto the RWT main, rapid corrosion failure at anodic points where stray current exits the pipe continuously would be expected. It's estimated that 20 pounds of metal loss can result each year per amp of continuously exiting stray current at the anodic point of WSP or DIP mains. The point at which the RWT main crosses beneath both gas mains invited special CP design: a 125 mil HDPE dielectric membrane above the RWT main and a new test station at each gas main with a direct current (DC) decoupling device providing DC isolation and grounding up to a 3 volt threshold. This decoupling device seeks to eliminate DC exiting the gas mains' CP systems.

When potentially corrosive environments are detected, protection schemes pair exterior pipe surface protection to an active impressed current CP system or passive galvanic anode CP system. Generally, exterior protection feature polyethylene wraps and bonded dielectric coatings. Cost and reliability must be carefully weighed, as prices for bonded coatings sharply vary between WSP and DIP manufacturers. In contrast to WSP fabrication, which is conducive to shop application, DIP manufactures typically use a third party supplier and applicator with pipe changing hands from the pipe manufacturer to the coating vendor—this shift of liability signals significant warranty restrictions for DIP manufacturers when conforming to a bonded dielectric coating specification. The approximate incremental cost for DIP with a bonded coating is estimated at over 10 times that of polyethylene encasement. For a 5.25-mile long pipeline, this differential scales to a considerable sum. As extensively studied by the National Academy of Sciences based on miles of ductile iron water, gas, and oil pipelines, when it comes to reliable corrosion protection of DIP mains, polyethylene wraps enhance longevity when compared to bare pipe, but bonded coatings are generally viewed as superior in this regard. This is a hotly contested issue in the transmission main marketplace. Based on convincing large empirical data sets of transmission main condition assessments and failure investigations, e.g. 20- to 40-mile 54- and 60-inch pipelines, practically no failures were evident when a bonded coating was paired with CP. In turn, this informed the Engineer's decision to specify three allowable bonded coating systems—3 layered tape, polyurethane, and polyolefin—and galvanic anode CP, which centrally features sets of 20-pound magnesium anodes buried adjacent to the pipeline at trench bottom. During construction, the Contractor sought to raise anode depth to pipe springline out of concerns that the anodes would hinder satisfactory backfill at the pipe haunches, which in turn, may generate long-term structural vulnerability to external loading, i.e. pipe deformation; however, anode placement at trench bottom was enforced, as justified by the need for anodes to be in moist soils in order to yield sacrificial protection current to pipeline cathodic surfaces.

Field Welded Joints

Implementing field welding in full compliance with the Engineer's specifications was paramount importance, particularly since this was the Owner's first WSP transmission main project. In fact, field welded joints garnered the focused attention of the Owner's executives. Personnel directly responsible for field welding were to be qualified in accordance with American Welding Society (AWS) D1.1 for structural steel or ASME, and included: the Contractor's field welder, the Contractor-hired independent AWS Certified Welding Inspector (CWI) who executed magnetic particle testing (MPT), and the Owner/Engineer's welding specialist. The project gains maximum benefit if the field welder performs in full and strict accordance to the specifications; accordingly, a submittal was required on the Welding Procedure Specification (WPS)—standards for double- and single-lap welds to which the field welder must demonstrate the ability to weld. Given that the steel pipe coil was Grade C material and since there's no American Welding Society (AWS) pre-qualified WPS for this material, the WPS needed to be expressly certified for Grade C. The Contractor produced certified WPS by invoking the steel pipe manufacturer's welding procedure. Upon Engineer's approval of the WPS, the field welder demonstrated the ability to weld to these standards by submitting test results conducted at and witnessed by an inspection services laboratory for:

1) 1-inch thick "V" groove welding of Grade C material with the electrode's current, voltage, and travel speed ranges tabulated; this WPS automatically qualifying him for field fillet welds as the skill demanded by groove welding surpasses that of fillet welds; results from tensile break and bend tests were offered as the weld Procedures Qualification Record (PQR); tested specimens' ultimate strengths exceeded 60 ksi and ruptured in the parent base metal, rather than the weld material; therefore, this PQR was accepted; and
2) 5/16-inch fillet weld of 48-inch Grade C pipe, with the electrode's current, voltage, and travel speed ranges tabulated.

Strength of Singe-Lap Joint Welds: While the specifications refrained from prescribing restrained joints be single or double fillet lap welded, the Contractor choose to exclusively pursue interior single lap welds, with the exception of double lap welds of carrier pipe at trenchless crossings. The Engineer pre-evaluated the strength of single lap welds according to the American Welding Society's (AWS) Effective Throat Length methodology, which accounts for of the project's test pressure, pipe wall thickness, and river water temperature range. This methodology demonstrated the allowable longitudinal force in the pipe wall exceeded the maximum anticipated value—the anticipated value is the sum of thermal and Poissons' stresses, while the allowable value is predicated on a grade C electrode producing a stick weld of 21,000 psi since weld material must afford yield and ultimate strengths equal to the parent metal. A complete outlook requires consideration of two alternate methods for assessing strength of single fillet welded joints: i) ASME's Pressure Vessel (PV) Joint Efficiency; and ii) Joint Eccentricity. As long as the fillet weld is the full leg dimension, the AWS methodology is acceptable for evaluating thrust loads. The project's specifications are consistent with this approach by requiring steel coils be of a wall thickness with a zero minus tolerance. The ASME PV approach scales the steel yield strength down by an efficiency coefficient, independent of the AWWA – prescribed maximum wall stress of 50% yield. The Joint Eccentricity approach is central in the current debate of single fillet weld strength since it is focused on the bending moment in the weld material resulting from the inherent eccentricity of the thrust load path between the pipe bell and spigot via the fillet weld. This approach imposes the bending rotational stress, creating a significant load for the fillet weld to withstand; for typical pressures, this moment-induced stress causes total wall stress to exceed allowable levels. This vulnerability has prompted some design engineers to strictly specify double lap weld restraint for all steel pipe projects.

Contractor Elects Weld After Backfill for Single Lap Interior Welded Joints: To not hamstring the Contractor by prescribing means and methods—which helps derive market value in a competitive bid—the Engineer intentionally did not specify the pipe laying-backfill-joint welding sequence, i.e. joint welding after or before backfilling the pipe. In the context of the Contractor's planned daily pipe laying production rate of 200 to 300 feet, joint welds are performed a variable length of time after the pipe is laid and backfilled; consequently, welding occurs after the heat shrink sleeve is installed. Thus, the heat shrink sleeve's ability to sustain a 100-year service life depends on its ability to dissipate heat borne from interior joint lap welding. This sleeve is installed on the exterior of each joint to shield the pipe from soil corrosion; it's noted that the majority of the 5.25-mile alignment traverses through aggressive soils. During submittal review, the Engineer ensured the sleeve manufacturer was aware that interior joint welding would be performed with the sleeve previously installed, since an often used, less costly sleeve is strictly for pipe that has not yet been backfilled; in particular, the sleeve's engineering properties sustain high temperatures—namely, its softening point is 401 degrees F. The Engineer's field quality control inspector was alerted to an incorrect grade B electrode used by the Contractor's welder prior to the first field joint weld; once this error was recognized and communicated, the Contractor's welder demobilized and returned with the correct grade C electrode. It's noteworthy that grade B steel features an average of 36 ksi yield strength, while grade C features an average of 42 ksi.

Magnetic Particle Testing (MPT) of Welded Joints: Quality control and Contractor oversight of the more than 700 field pipe joint welds was a major point of emphasis for the project. With the weld material magnetized by the electromagnet instrument, small leakage in the magnetic field develop across cracks in the weld material. A colored powder of magnetized particles placed by the inspector along the joint are concentrated at and held by these leakage fields, enabling the inspector to visually determine the presence of cracks. From the MPT instrument manufacturer chosen by the inspector, Parker, the D400 model used on this project is a two-pole electromagnet that applies an AC or DC induced magnetic field in the joint weld material. The physical connection of the pipe and instrument creates a magnetic flux path, with the weld material becoming highly magnetized. From within the pipe, the inspector passes the instrument circumferentially along the joint to detect defects (surface and near-surface cracks) in the joint weld material. AC magnetization is advantageous for surface breaking cracks, while DC magnetization is best for near surface cracks. Welds for this project were tested with both AC and DC magnetization. The instrument is powered by an 120 V AC chord routed through 3-inch weld lead outlets or manway outlet appurtenances, and either passes 6 amps of AC current onto the pipe weld material or converts the AC to DC via internal electronics as activated by the operator. Specifications called for carrier pipe double weld joints at trenchless crossings, i.e. running within casing pipe, to undergo MPT on the interior and exterior welds; as a leak within the casing would go undetected, carrier pipe joints at trenchless crossing were to be double lap welded. The Contractor initially disputed the interior and exterior MPT, claiming testing exterior welds would impede pipe lay production; the Engineer choose to enforce the double weld MPT specification. Specifications required a Contractor-hired MPT technician, certified at the proper American Society for Nondestructive Testing Central Certification Program (ACCP) non-destructive testing MPT level.

Seismic Loading Increases Axial Stress in Pipe Wall: During design, the incremental increase in pipe wall stress due to seismic loading was assessed. Within the quarry storage water banking framework, approximately a third of the pipeline alignment is offset from a planned future quarry by nearly 50 feet. Consequently, future quarry blasting loads are expected to impart onto the pipe. Ground accelerations translate into strain in the traverse direction of the pipeline; through Poissons' relationship, this imposed strain manifests as an incremental rise of axial stress in the pipe wall. This axial, or longitudinal, stress was factored into our assessment of the strength of single lap welded joints.

Construction Schedule: WSP Production Rate Verses Typical DIP Production Rate

The Contractor's construction schedule is predicated on a daily pipe lay production of 250 feet is planned, far exceeding the typical DIP rate of 100 feet. This creates value for the project overall by helping reduce delay risk imposed by unanticipated changes, namely changed field conditions. For example, initial pipe laying has been hindered by repeated problems with the onsite "rock crushing" operation through which native heterogeneous excavated material undergoes physical processing to produce a more compactable backfill. The ensuing delay risk is mitigated by the 250-foot daily lay rate afforded by comparatively long steel pipe segments—typically 50 feet.

Special 90 Degree Elbow Fittings

In three locations, constrained easement prevented use of standard 90 degree horizontal steel pipe elbows, which according to M11 are standardized with a minimum radius of 2.5 times outer pipe diameter, i.e., 2.5 D. This issue emerged during development of the steel pipe lay schedule by the steel pipe manufacturer—as DIP 90 degree bends are of a much tighter geometry, with a radius of about 0.7 times pipe diameter, the need for specially fabricated steel pipe elbows was not realized during alignment design, since it was based on DIP. The easement afforded the lay of 90 degree elbows with: i) 1.5 D; ii) 1.25 D; and iii) 0.75 D. Stress intensification calculations for the elbows' inner diameter walls were performed for these three geometries under a pressure-thrust loading condition. Elbow geometry must also consider welding borne stresses as the segment length between shop welds reduces. Stress calculations proved the specified wall thickness (0.3125 inches for 42 -inch pipe) satisfactory for the 1.5 and 1.25 D cases, while the manufacturer chose to roll 0.5 inch plate for the 0.75 D elbow. This highlights an unforeseen advantage of otherwise unnecessarily thick walled pipe.

Conclusions

Design and construction of this Northern Virginia 100 year service life, non-redundant, raw water pipeline has required the pinnacle level of evaluation, analysis, and quality control. The material evaluation applied to this project contrasting WSP versus DIP, predicted WSP to be less costly based on the wall thickness and bonded coating specifications. This prediction was affirmed as all received bids were for WSP. During design, a series of material-specific considerations was captured by the Engineer's bid documents, including: i) vault systems and appurtenances unique to WSP and DIP; ii) alignment geometry to accommodate joint deflection capabilities; iii) transient modeling to size and position anti-surge valves and surge tanks at the high service pump station, which incorporated separate WSP and DIP celerity values; and iv) a robust passive sacrificial anode cathodic protection system to counter native soil and stray current –induced corrosion. Horizontal and vertical geometry design was completed in particular detail given the high land values associated with the fully developed geography traversed by this pipeline. The utility corridor character of its alignment introduced significant stray current corrosion potential from parallel high pressure gas mains. Construction quality control has been, and continues to be, essential for this high profile project. For WSP pipelines, the Owner and Engineer must be experienced with magnetic particle testing of interior and exterior field welded joints. An impressive number of field welds are required for this project as the alignment features a myriad of horizontal and vertical geometry changes to position the pipe within the existing utility corridor. Moreover, the strength of single lap field welds should bear on decision makers when designing and constructing WSP pipelines.

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Lessons Learned in the Design, Manufacture, Shipping, and Installation of the 108-inch Integrated Pipeline (IPL) Section 15-1

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Abstract

The Tarrant Regional Water District (TRWD) and the City of Dallas Water Utilities (DWU) are engaged in the planning, design and implementation of a 350 MGD raw water transmission system. Referred to as the Integrated Pipeline Project (IPL), the system consists of approximately 150 miles of 84- to primarily 108-inch diameter pipeline, a 5 mile 120-inch diameter tunnel, six 100–350 MGD pump stations, one 450 MG balancing reservoir, and ancillary facilities. When complete, the "integrated" system will provide a critically important source of water for the rapidly growing Dallas-Fort Worth Metroplex for the next five decades and beyond. After years of study, followed by design and construction of a 2 mile, 108-inch steel water pipe demonstration project, the Line J Project, the first segment of the pipeline, Section 15-1, bid in February 2014 and was awarded in March 2014. Steel pipe was chosen for the construction. Pipelines of this diameter and length are not common and come with their own set of challenges from the design, manufacture, shipping and installation standpoints. This paper will review design of this piping material and the practical aspects of furnishing and installing a line of this magnitude, with particular emphasis on *lessons learned* on both the Line J and the Section 15-1 Projects from the view point of the Owners, Engineer and Manufacturer.

INTRODUCTION

The Tarrant Regional Water District (TRWD) with the City of Dallas Water Utilities (DWU), are currently engaged in the construction of the beginning phases and planning, design and implementation of a 350 MGD raw water transmission system, which will run across north central Texas from Lake Palestine to Lake Benbrook, with connections to Cedar Creek Reservoir, Richland Chambers Reservoir, and a Dallas delivery point. Collectively, the system consists of approximately 150 miles of primarily 108-inch pipeline, with some sections of 84-inch diameter pipe, a 5 mile 120-inch diameter tunnel, six 100–350 MGD pump stations, one 450 MG balancing reservoir, and ancillary facilities. The joint program developed by TRWD and DWU is called the Integrated Pipeline Project (IPL). The project-location in shown in Figure 1.

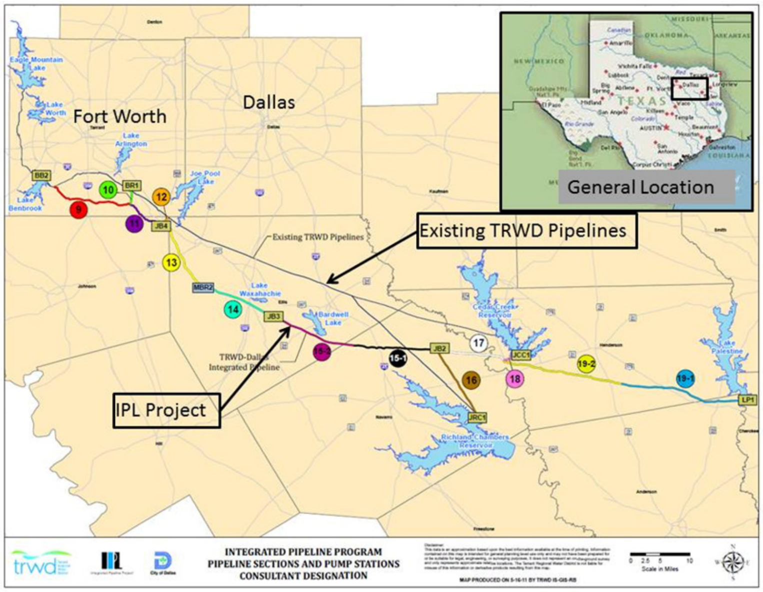


Figure 1: Project Location

TRWD and DWU currently provide drinking water to an estimated 4.4 million people. Based on developments and updates of the City of Dallas and Texas Water Development Board long range planning studies conducted in 2005-2006, it is predicted that population and water demands are likely to double in the next 50 years. The IPL project is developed to provide an additional 350 MGD supply to meet these growing needs. The project is being developed in five distinct phases with completion of Phase 1 (70 miles of 84-inch to 108-inch pipeline, a 350 MGD booster pump station, three interconnection facilities, a 450-MG terminal storage balancing reservoir, and ancillary facilities) in 2018, and Phases 2 thru 5 by year 2035. Design of pipelines and facilities also took into consideration the potential for future expansions.

SECTION 15-1 OVERVIEW

Section 15-1 of the IPL project consists of a portion of mainline pipeline, a 5,400 sq-ft interconnect facility, and the lowering of an existing 90-inch water line, all to be constructed as part of Phase 1. The project spans from the site of future pump station JB2 west to the interface point with section 15-2 all within Navarro County, Texas, Figure 2.

Figure 2: Location of Section 15-1

The pipeline consists of approximately 15.5 miles of 108-inch diameter pipe. It also includes seven tunnels (90-ft to 450-ft drive lengths) crossing various highways, a Union Pacific Railroad crossing, existing gas utility and a creek. A 5,400 sq-ft interconnection facility is located approximately 6 miles west of the future JB2 pump station. The facility includes twelve 42-inch butterfly valves and 78-inch interconnection piping. This provides pressure reduction and connection from the new IPL system and the existing TRWD 90-inch diameter PCCP Richland Chambers pipeline. The overall project included a relocation / lowering of approximately 2,000-ft of the existing Richland Chambers water pipeline with 90-inch steel pipe which was constructed as part of a Construction Manager at Risk (CMAR) project in advance of the mainline project.

Project design was conducted from early 2011 until late 2013; design commenced with a route alignment refinement, continued with coordination among other program consultants (topographical and land survey, environmental/archaeological permitting, geotechnical investigation and other detail design teams), advanced with preliminary engineering and ended with final detail design of improvements required to complete the project.

The project was bid from Dec 2013 to February 2014 using a Competitive Sealed Proposal process. This process allowed selection of successful proposer considering the best value to project owner. The project bidding included four options for contractors to select in offering their proposal – options included choice of pipe material (steel pipe or prestressed concrete cylinder pipe) and choice of embedment type (compacted granular material or native soil Controlled Low Strength Material, CLSM). Six proposals were received ranging from $92.0 million to $127 million. A construction contract was awarded to Garney Companies, Inc. of Kansas City, MO in March 2014. Northwest Pipe Company of Saginaw, TX provided the steel pipe.

Construction began in June following mobilization and preliminary activities. The Contractor utilized three pipe headings to complete mainline installation in April 2015. Completion of appurtenance build out continues with anticipated project completion by June 2015.

DESIGN

While the design of steel pipe might appear to be very straightforward, when utilizing AWWA M11 as a design guide, the steel pipe design for the IPL pipeline was quite complicated. The initial IPL program design criteria manual and specifications were compiled via a joint effort of their initial consultants on the project. These two documents were given to all of the individual design teams for the eight pipeline contracts that make up the IPL pipeline. The original thought process was that each of the eight sections would therefore have similar pipe material specifications to assure equal performance and keep the pipe material competition level. However, early on in the specification process, it became clear that each consultant desired to utilize their own company “standard” specification for the pipe material options. Moreover, there were discrepancies between the design criteria manual requirements and the pipe material specifications that the consultants were instructed to use. Because of the discrepancies between the two documents and the desire of each consultant wanting to utilize their own familiar pipe material specifications, the IPL program management stepped in and demanded the use of their original standard specifications unless they contained a “fatal flaw” that would prevent any of the individual engineers on each segment from signing and sealing the specifications for their section.

After resolving several rounds of "fatal flaw" issues in the design criteria manual and specifications, two unified documents were developed for the consultants to use on their individual sections. As the LAN team was designing Section 15-1, which would be the first actual pipeline segment to be bid, awarded and built, the unified documents formed the base for the design and specification of this initial section.

108-inch diameter pipe would be considered "large diameter" by any pipeline designer. As such, design or manufacturing elements utilized in typical 24-inch to 60-inch diameter pipe might not scale very well to this large size.

One example of this scaling up process is the placing of the cement mortar lining on the interior of steel pipe via the shop centrifugal spinning method. While it is routine to shop spin cement mortar line 24-inch through 96-inch diameter pipe, there have only been very limited amounts of steel pipe produced greater than 96-inch diameter with shop spun cement mortar lining that could be documented. After researching the available projects that had been supplied by the various steel pipe manufacturers with shop spun cement mortar linings on 96-inch and larger diameters, it was decided to specify shop spun cement mortar lining for diameters up to and including 108-inch. But, as a change to the AWWA C205 requirements for cement mortar lining, the IPL program specified the lining to be ¾-inch thick. This additional thickness was added to address water chemistry and quality issues to meet the 100 year design life. The thicker cement mortar lining also slightly enlarged the outside diameter of the steel cylinder to account for this greater lining thickness and still provide the 108-inch inside dimension after lining. Additionally, for any nominal diameters larger than 108-inch, the cement mortar lining was specified to be field applied after installation.

Pipeline Section 15-1 contained internal pressure classes up to 225 psi working. When using the typical grade 42 steel (minimum specified yield strength of 42,000 psi), and the program-specified minimum thickness for handling (D/t) ratio of 230, a resulting pressure class of approximately 182 psi is derived. Therefore, for the higher pressure classes contained in this section, an increased steel cylinder thickness would be required. Recognizing that the cost of the pipe is the major driving force behind the overall cost of the project, grade 46 steel was suggested. Increasing the grade to 46,000 psi minimum specified yield strength also increased the working pressure that the minimum cylinder thickness could be designed to hold up to approximately 200 psi. This slight increase in grade of steel would result in 9.5% cost savings in the steel for the class 200 and 225 psi pipe. Nationally water pipe projects with steel grades over 42,000 psi yield strength that are shop-applied cement mortar lined are somewhat limited and they are typically in the smaller diameter range. TRWD did however have considerable successful experience using 46,000 psi minimum yield strength steel on their multi-mile 96-inch and 84-inch Eagle Mountain projects. After additional discussions between pipeline design professionals, including those on the AWWA Steel Pipe Committee, along with the program's consultants and the pipe manufacturers, it was decided to allow the use of 46,000 psi yield steel on the project. Safeguards were written into the specifications to limit cement mortar lining damage during handling, transport and installation. Moreover the specifications required a field hydrotest pressure in excess of the specified working pressure classes, another safeguard for the owner to receive a pipeline with the desired 100 year design life.

Smaller diameter steel pipe (with sizes up to and including 72-inch diameter) typically have a push on rubber gasketed O-Ring joint for the standard joint and a single welded lap joint for the restrained joint. Since the 108-inch diameter exceeded that of any manufacturer's standard O-Ring joint, a single lap welded joint was chosen for the desired field joint. Single welded lap joints are designed to function with surge pressures of 150% of the working pressure or Pipe Class and for full thrust conditions. Deep welded bell joints were specified at regular intervals to prevent the buildup of excessive thermal stresses during installation. Another safety factor against this thermal stress buildup was to allow the Weld-After-Backfill (WAB) procedure, which also results in cost savings to the owner. Because both of these limit the total thermal stress in the pipeline, a full thickness, single-welded lap joint was required to satisfy the design. As with any pipeline, select field joints will be flanged along with electrical isolation joints at the proper connection locations.

Several options on the bedding and backfilling of steel pipe were offered for both the typical cover conditions (5-ft minimum up to 18-ft of fill over the top of the pipe) in the trench details. In general existing soils were clay or fatty clay materials along the 15 mile Section 15-1 alignment which is typical of the soils along the majority of the IPL project. The IPL program had invested in testing various bedding and backfill scenarios that included large diameter steel pipe embedment in various materials along with testing the on-site manufacture of CLSM out of the native soils. These demonstration projects showed that native soil CLSM provides an excellent embedment and structural support for flexible pipe materials. However, these research and development projects also demonstrated a much slower production rate in pipe laying, embedment and backfill than what can be realized when using more standard construction methods. This was proved out by the actual bid results as Garney, and all but one bidder, used the specified "granular embedment option C" in their proposal, Figure 3.

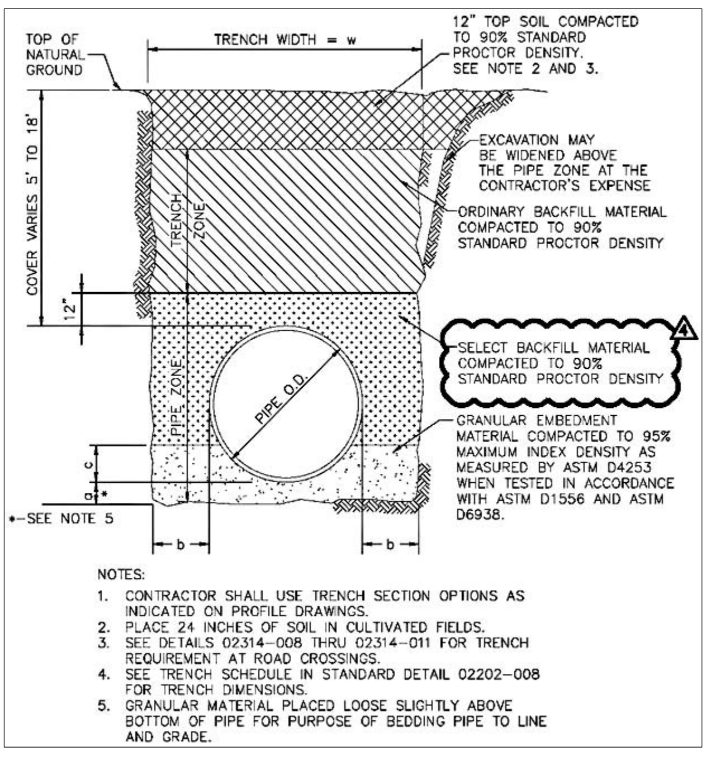
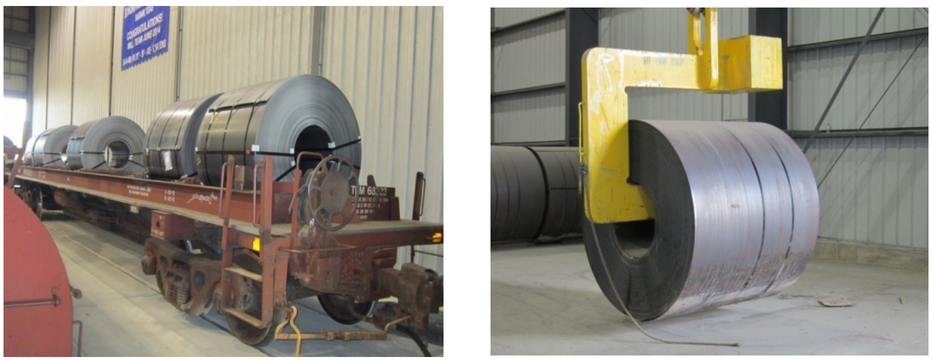


Figure 3: Section 15-1, Trench Section Option C

In Trench Section C, Figure 3, the "c" dimension was 0.7D, with the "a" dimension = 6-inch and the "b" dimension = 24-inch for a total trench width of OD + 4-ft. The granular embedment is classified by the specification as cohesionless material such as crushed stone, pea gravel, river rock, or gravel embedment (GW, GP) or sand embedment (SP, SW), 100% passing a ½-inch sieve all compacted to 95% maximum density.

STEEL CYLINDER MANUFACTURE

Steel water pipe cylinders were manufactured per AWWA C200 standard and project specifications. The cylinders were made from coil steel, Figures 4a and b that were pulled through buttress roles, edges prepared for welding, and then spirally free formed into a cylinder that is submerged arc welded both on the inside, Figure 5a, and the outside, to produce full penetration welds.



Figures 4a, b: Coil Steel

The cylinders were then cut to typical 50-ft sections. Ends were precisely prepared for lap welded field jointing, Figure 5b. Pipe cylinders with prepared ends were then hydrostatically tested, Figure 5c, to a pressure that engaged 75% of the minimum specified yield strength of the steel. For the 108-inch cylinders on Section 15-1, the applied hydrostatic pressure was 267 psi. This hydrostatic pressure is typically equal to the design surge pressure and in effect provides a "proof of design" test against leaks for each section of pipe.

Figures 5a, b, c: Interior Submerged Arc Welding, Expanding Bell-End of Cylinder, Hydrostatic Testing of 108-inch Cylinder to 267 psi

Quality Control specified by TRWD for the steel cylinders were stringent and matched closely with Northwest Pipe Company's ISO 9001 standards. These requirements included material selection and verification, welding, dimensional tolerances and all forms of destructive and non destructive testing. A "lesson learned" is that permanent marking or stenciling of the pipe cylinder, Figures 6a and b, can be used to identify each section of pipe as installed in the trench and tie the specific pipe back to the pipe manufacturer's QA records. TRWD would then be able to incorporate this installed information into their GIS and SCADA systems so as to be able to identify each section of pipe by location and also to be able to trace all material, dimensional and manufacturing QA records in the future as needed.

Figures 6a, b: Stenciling on Interior on Spigot-End of Each Cylinder

CEMENT MORTAR LINING APPLICATION

Cement mortar lining was applied per AWWA C205 and project specifications. Details of the cement mortar specifications were discussed previously in the Design discussions. Specifications required ¾-inch minimum thickness of cement mortar. A number of lessons learned were identified during the project, listed below.

1. ¾-inch thickness can be applied in 108-inch and larger pipe with modifications to the cement mortar lining equipment and amount of spin time. Shrinkage cracks were no greater than that of AWWA C205 for ½-inch lining, and well within allowances. The extra lining added considerable weight to the pipe. The extra ¼-inch thickness was necessary due to the aggressive chemistry of the raw water that the pipeline would be conveying.
2. TRWD previously built the 108-inch Line J demonstration project where it was shown that a number of experienced steel water pipe suppliers could manufacture, handle and ship 2 miles of 108-inch cement mortar lined and polyurethane coated pipe over great distances. The qualified manufacturers provided considerable competition for pipe material pricing on Section 15-1 as the project that came in approximately 20% under budget estimate. Figures 7a and 7b show the lining of 25-ft and 50-ft sections of cylinder, respectively.
3. Stulling or bracing of the 108-inch cement mortar lined steel pipe received considerable prior study. The primary purpose of the stulls is to keep the pipe and cement mortar lining from excessive "flexing" during handling, shipping and jointing the pipe in the trench. From the Line J project experience, IPL settled on a stulling arrangement for the 108-inch pipe that was in excess of what is typically used in smaller diameter pipe but appropriate for the fact a 50-ft section of 108-inch pipe weighs in excess of 43,260 pounds. The standardized stulling configuration worked well during the installation of the Section 15-1 project. Figures 8a and 8b show the finished interior surface of 108-inch diameter cement mortar lined cylinder prior to stulling, and installed 6-point stulling, respectively.


Figures 7a,b: Cement Lining of 25-ft Long Cylinders, 50-ft Long Cylinders


Figures 8a, b: Finished Interior of Lined Pipe, Installed 6-Point Stulling

POLYURETHANE COATING APPLICATION

Polyurethane coating was applied to pipe per AWWA C222 standard, modified to minimum 35 mil thickness. Lessons learned are listed below.

1. Original Line J project specifications required extended time between applying polyurethane and ability to holiday test the coating during application. The Holiday test, Figure 9a, provides a DC current directly to the coating at a voltage of 100 volts per mil of specified coating thickness, or in this case, 3500 volts. Any voids or pinholes (referred to as *holidays*) in the coating would be identified and repaired immediately. It was learned during pipe making for the Line J project that per manufacturer's recommendations, the holiday test could take place while the pipe was still on the "lathe" and within a few minutes after application. This avoided "over handling" of the pipe for the purpose of holiday testing.
2. To reduce the temperature of the polyurethane coating during the hot Texas summer days where temperatures can easily increase above 100 deg. F, it was decided to utilize an "off white" polyurethane coating color in lieu of the standard Northwest Pipe Company dark blue coloring. Lower temperatures may have long term benefits for the pipe coating's desired 100 year design service life.
3. Aromatic polyurethanes used for buried pipe coating can be expected to "chalk" or lose the top 2-3 mils of coating thickness during extended ultraviolet (UV) exposure to the sun. Often, applicators apply additional thickness to make sure the specified minimum 35 mils of

coating are applied. While the additional thickness "is not by design," the additional mils that were applied appeared to more than address any coating loss concerns from chalking. It should also be noted that the specified minimum coating thickness of 35 mils is considerably greater than the minimum 25 mil guideline in AWWA C222.

Figure 9a, b: Holiday Testing, Coated Pipe Just Off the Lathe

SHIPPING AND HANDLING

Shipping and handling of 108-inch pipe warrant special precautions due to the dimensions, weights and safety considerations. Following is a list of the lessons learned in this arena.

1. 50-ft sections of pipe were shipped with 3 sets of curved and padded bunks on the truck bed that corresponded to the placement of the interior stulls. Ends of the pipe were "capped," Figure 10, which helped continue the cement mortar lining curing or hydration process, which in turn limited the cracking of the cement mortar during shipping and handling to meet requirements of AWWA C205.

Figure 10: 50-ft Section of Pipe on Truck w/ End Cap

2. Pipe should be unloaded with nylon straps or equal and with two properly placed pick points. This is accomplished with spreader bars, Figure 11a, two pieces of equipment lifting the pipe simultaneously or by other means. Pipe should be placed on earthen berms, Figure 11b, or similar to facilitate picking pipe up to install. End caps should remain on until installation.

Figures 11a, b: Nylon Straps and Spreader Bar to Move Pipe, Earthen Berms for Pipe Placement

3. Handling the pipe during installation in the trench, Figure 12, again requires nylon straps or equal and again with properly placed pick points. Stulls should remain in pipe during this operation to limit "flex" of the pipe cylinder and cement mortar lining and to aid in keeping pipe as round as possible during joint make up. This facilitates joint fit up and equalization of the gap for full fillet field welding.

Figure 12: Pipe Handling in Trench

INSTALLATION

There are likely a number of lessons learned from both the TRWD Line J project and Section 15-1 project's installing contractors could share but this paper will focus on just a few key lessons learned.

1. The proposed use of CLSM as a cost effective alternative to imported granular materials on Section 15-1 proved not to be the case. Key element was the time required for excavation,

staging, mixing of materials and cure time. This has been the case on the following IPL projects bid to date.

2. Attention should be paid to the haunching of granular material under any large diameter pipe, whether flexible or rigid. It is important that granular material be placed in the "pie shaped" haunch section under the pipe all the way to the invert of the pipe. Voids under the haunches have potential to be problematic for all pipe materials. Use of granular material, placed in controlled lifts combined with suitable compaction such as a "compaction wheel" is producing good results on Section 15-1, Figures 13a and 13b.

Figures 13a, b: Compaction Wheels, Application of Bedding Material in Haunch Zone in Lifts and Using Compaction Wheels

3. Heat Shrink Sleeves are applied after joint is made up when using the specified Weld-After-Backfill (WAB) method, Figure 14a. Heat shrink sleeves are industry standard practice whose installation procedures are well known. Attention should be taken to make sure that during the backfill process, the haunch zone is completely filled in the area of the bell hole (dug to facilitate joint make up and installation of the heat shrink sleeve). It was found that due to the size of the bell hole and the large radius of the pipe, the standard practice for backfill described above would not always completely fill the haunch zone of the bell, Figure 14b.

Figure 14a, b: Application of Heat Shrink Sleeve, Voids in Bell Holes after Completion of Backfilling

Since the bell hole is in the heat affected zone for the WAB process, the haunch zone should be completely filled. It was learned on Section15-1 that a certain amount of haunch material needed to be placed by shovel or by vibration equipment while the bottom crew was still protected in the trench box. This practice has been proven to work well by excavation of installed and welded joints to inspect material placement under the haunches in the bell hole area and to test the heat shrink sleeve itself for any damage to the outer backing. Results have been good with modified procedure.

4. 108-inch Polyurethane Coated steel pipe in casings or tunnels --- The Line J project used both commercially available casing spacers and "mortar bands," Figure 15, applied directly to the polyurethane coated pipe at the shop. Both the casing spacers and mortar bands are designed to act as "sleds" to facilitate pushing and or pulling the 108-inch pipe into the casing or tunnel. They are also designed to keep the casing from contacting (or shorting) to the 108-inch carrier pipe. Shorting of the bare casing pipe or tunnel liner plates would require the cathodic protection system to protect the uncoated casing pipe which will drive up the cathodic protection requirements significantly. Experience on the Line J project and Section 15-1 has shown that both casing spacers and mortar bands require care in the installation of pipe within tunnels to avoid damage to these features.

Figure 15: Two Mortar Bands on 25-ft Section of 108-inch Polyurethane Coated Steel Pipe

CONCLUSION

TRWD and DWU engaged the LAN team to design Section 15-1, the first segment of the IPL Project. Design was based on a 100-year service life. Bids were taken in February 2014 on the 15 mile pipeline and steel pipe was selected. The pipe was supplied by Northwest Pipe Company and the Garney Construction installation team is scheduled to complete installation by June 2015, almost a full year ahead of schedule. The lessons learned on this segment, which were discussed in this paper, have served to enhance or improve the overall quality of the pipe material and installation. To date, the bid prices for not only Section 15-1 but also for the other IPL projects have been much lower than the project estimates, and steel pipe with granular bedding has been chosen for all projects. TRWD and DWU are pleased with the work of their consultants, researchers, contractors and suppliers and expect the IPL project to continue on scheduled and provide raw water to the Dallas-Fort Worth Metroplex for many decades to come.

Steel Water Transmission Mains in Liquefiable Soils in Hillsboro, Oregon, Planning Considerations

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Abstract

Soil liquefaction has long been recognized as one of the greatest hazards for the integrity and performance of water transmission systems during and after major earthquakes. The effects of soil liquefaction include relatively large magnitudes of permanent ground deformations (PGD) in the form of vertical settlements and horizontal movements (lateral spreading). The effects can also include loss of soil strengths and flotation. Pipeline performance during previous major earthquakes showed that flexible, durable, strong pipes and joints can tolerate some degree of the liquefaction induced deformations. Welded steel pipe is considered one of the better seismic performing pipes. In some degrees, steel pipe can withstand plastic yield but still maintain integrity and service during and after earthquakes. For steel water transmission system planning and design, overall assessment of liquefaction hazards, appropriate route selection, pipe thickness and weld selection are the crucial elements. For deep pump stations and vault structures, foundation failure and flotation of the liquefiable soils should also be considered. At some locations pipe supports or ground improvements can provide some advantages in liquefaction mitigation. Flexible/extendable joints can also provide additional benefits. However, these additional mitigation features are typically associated with relatively high costs. In some significant PGD zones (i.e. forefront of the lateral spreading zone) with prohibitive mitigation costs, considerations can be given for planning emergency repairs and bypass at controlled, accessible location. This paper/presentation explores necessary geotechnical and liquefaction hazards assessments, steel pipe and joints evaluations, and mitigation method selections to optimize construction cost and seismic resiliency requirements for an approximately 30 mile long water transmission main project for the City of Hillsboro and Tualatin Valley Water District (TVWD) in Washington County, Oregon.

INTRODUCTION

TVWD and the City of Hillsboro, Oregon are developing their second major water supply system from the Willamette River in the City of Wilsonville, through more than 30-miles of large diameter water transmission pipelines to their service areas. This project is called the Willamette Water Supply Program (WWSP), and its location is shown in Figure 1.

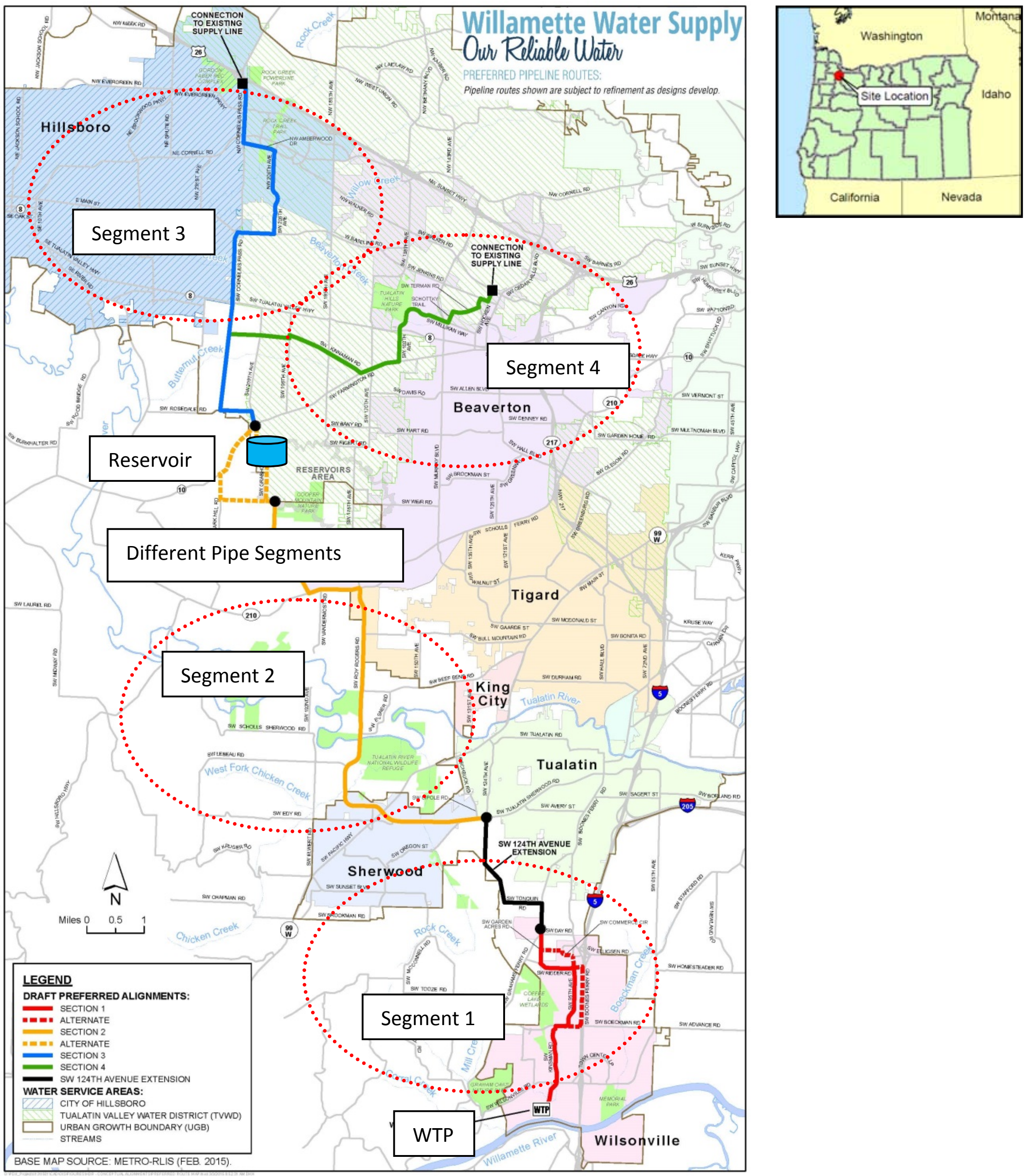

Figure 1. Map of Willamette Water Supply Program

Currently, WWSP is in the process of a siting study and evaluating different alignment options. One of the design requirements is that new system components shall be seismically resilient to withstand likely impacts from a design earthquake event with a return period of 2,475 years (2 percent probability of exceedance in 50 years). The scenarios for a 2,475 year design earthquake include a magnitude 9.0 Cascadia Subduction Zone earthquake (CSZ, for location see Figure2). The magnitude 9.0 CSZ was selected by the State of Oregon as the earthquake scenario of the development of Oregon Resilient Plan (ORP) in 2013.

WILLAMETTE WATER SUPLY PROGRAM (WWSP)

The WWSP consists of a new raw water intake on the Willamette River, a 110 MGD water treatment plant and pump station, a 72-inch 16-mile long welded steel transmission main, a 30 MG terminal reservoir, and two approximately 10-mile long 54-inch and 60-inch welded steel gravity pipelines connecting the new terminal reservoir to the existing transmission mains in the service areas. The approximate locations of these project components and pipeline corridor segments are shown in Figure 1. The WWSP pipeline segments are anticipated to be constructed mainly using open-cut method with a minimum soil cover of 7 feet and average trench depth between 11 to 13 feet below ground or roadway surfaces. Exceptions are for some deep crossings below river/creek, railroad, highway, and utilities, where trenchless methods are considered for pipe installation.

The majority of the selected pipe corridor segments (main stem and gravity branches) will be laid within the existing or future roads of two counties and four cities. The corridor segments are also challenged by several major creek and river crossings, state highway and railways crossings, vicinity of protected wildlife refuge areas, wetlands, hazardous waste sites, and major sewer, water, gas, electric and oil transmission mains within the existing public rights-of-way.

Each pipe corridor segment consists of three different pipe sub-routes that will need to be further explored to more precisely elaborate additional site specific characteristics including more detailed location of existing utilities, soil characteristics, ground water levels, corrosivity of soils, potential for soil liquefaction, need for soil improvements and other sub-route specific seismic characteristics and concerns that need to be considered to meet pipe seismic resiliency requirements.

OREGON RESILIENCY PLAN

The 2013 Oregon Resiliency Plan (ORP) for Water and Wastewater Systems identifies that "Re-establishing water and wastewater service will be a crucial element in the overall recovery of communities after a Cascadia subduction zone earthquake. Water for fire suppression, first aid, emergency response, and community use, as well as water for normal health and hygiene, will be required soon after the event".

In addition, the 2013 ORP recommends that water-related industry associations and manufacturers evaluate the need for seismic design standards for pipelines, and encouraging The Oregon Health Authority (OHA) to include a seismic design

requirement as part of routine design review of water system improvements with the goal to ensure that seismic considerations are incorporated into designs for critical facilities.
The following Table 1 describes expected availability of potable water at different locations and operational system requirements for different water system components following the major earthquake.

Water availability at different locations and through different components	Availability of potable water in days					
	1-3	3-7	7-14	14-30	30-90	90-180
	Operational requirements in percentage (%)					
At water sources		20-30		50-60		80-90
Through transmission mains	20-30	50-60	80-90			
At critical facilities		20-30		50-60		80-90
At key fire points	20-30		50-60			80-90
At fire hydrants				20-30	50-60	80-90
Through distribution pipes			20-30		50-60	80-90

Table 1. Expected availability of potable water after major earthquake

PROJECT SEISMICITY

The seismicity of WWSP project area is subject to two major earthquake sources: (1) local, shallow crustal earthquake with relatively low magnitude (typically less than 7.0), and (2) CSZ earthquakes with large magnitudes (typically above 8.0 and with potential up to 9.0 or even 9.2). For a 2,475 year design scenario, CSZ earthquakes are considered to cause more damage than local crustal earthquakes.

CSZ earthquakes originate along at the interface of the Juan de Fuca and North American Plates (Figure 2), which is located approximately 20 to 30 miles beneath the coastline from north California to British Columbia. Recent seismological and geological researches (Atwater 1995 and Goldfinger et al. 2012) disclosed compelling coastline and ocean sediment evidences that CSZ earthquakes represent the most eminent seismic hazard in our region.

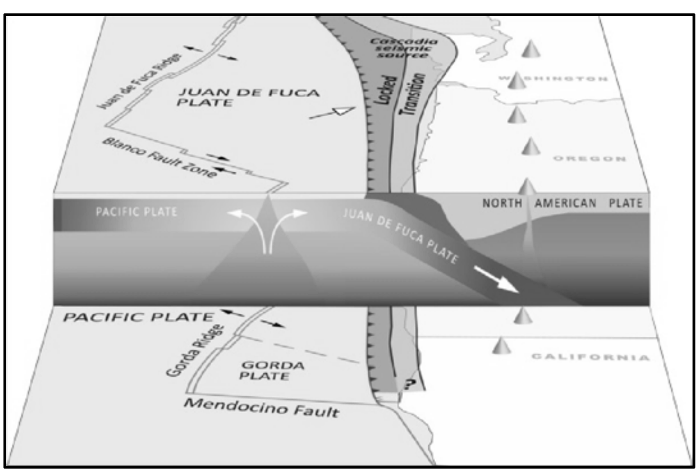

Figure 2. Cascadia subdaction zone location (DOGAMI 2010)

PROJECT GEOLOGIC CONDITIONS

The WWSP pipeline is located within the southeastern portion of an approximately 20-mile-wide valley of the Tualatin Basin. The bedrock at the base of the valley is the Columbia River Basalt which is also outcropped on the valley sides. The basalt is overlain by a few hundred feet thick Hillsboro Formation of ancient fine-grained clayey and silty sediments from more than 1 million years ago. Above the Hillsboro Formation, the valley ground surface is typically covered by "Willamette Silt", of approximately 100 feet thick sandy silt, silt, clayey silt and silty sand deposited 15,000 to 10,000 years ago by glacial flooding events. Also, within the past 10,000 years, Tualatin River and its many tributaries deposited 10 to 20 feet of fine-grained Recent Alluviums of soft clay, silt, fine sand, and organic soils above Willamette Silt along the low lying, narrow flood plains.

The average seasonal groundwater level is typically between 10 to 20 feet deep within the project areas with localized shallow groundwater (less than 5-feet in depth) near the floodplains of the Tualatin River and its tributaries. The Willamette Silt and recent river/creek alluviums are expected to have relatively high liquefaction potentials and will likely generate large PGDs. The soil liquefaction profile is typically extended from groundwater table to approximately 50 to 70 feet deep.

Considering the pipeline depths and the groundwater conditions, the WWSP pipeline will likely be located on the upper portion of the liquefaction zone or within the non-liquefiable soil crust. The general soil deposits and relative location of the WWSP pipeline in the Tualatin Basin are shown schematically in Figure 3.

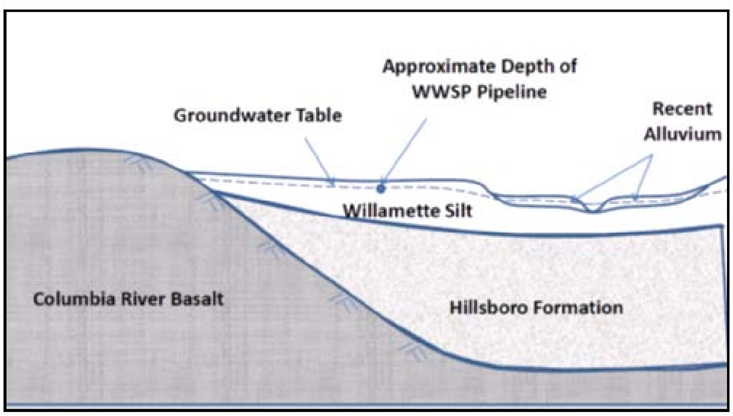

Figure 3. Generalized subsurface zone location and schematic section

POTENTIAL FOR LIQUEFACTION OF SOILS WITHIN PROJECT AREA

Based on published regional liquefaction maps from the State of Oregon (DOGAMI 1995, 1997 and 2013), a soil liquefaction potential map and general seismic hazards evaluation were developed for the WWSS pipeline corridors. A portion of this liquefaction map for Hillsboro and Beaverton areas (where the east and west gravity branches will be located) is shown in Figure 4.

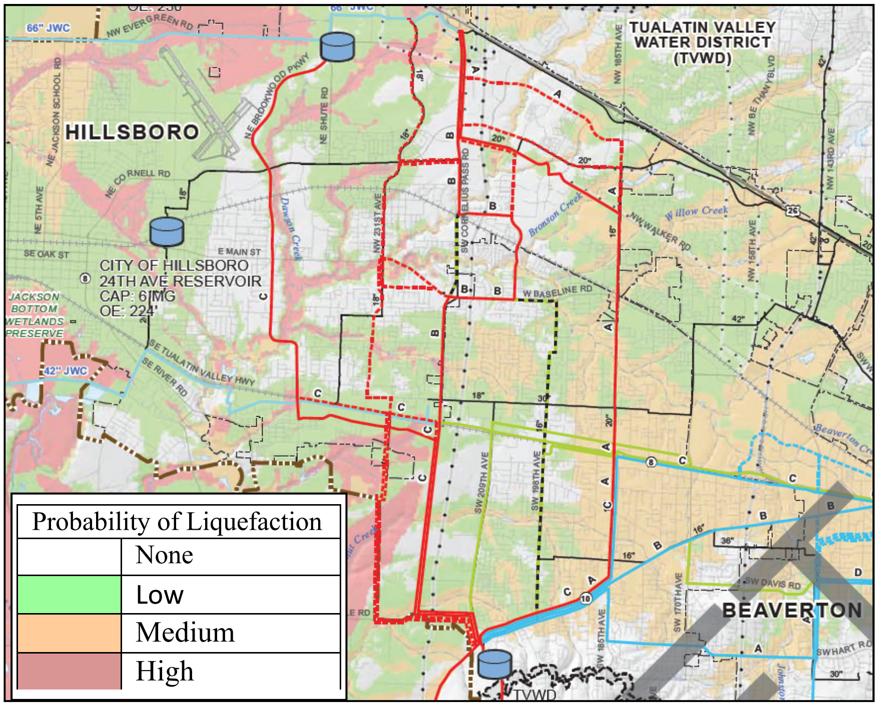

Figure 4. PGD Map Hazard Area - Segments 3 and 4

In general, soil liquefaction risk is generally low in the south and north sections of the main stem, but relatively high (medium to high risks) for the mid-portion of the main stem and the entire east and west gravity branches. Also the entire WWSP pipeline will very likely be exposed to different seismic hazard phenomena caused by transient loading, a shaking hazard caused by seismic wave propagation and the amplifications due to surface soil conditions and topography, and PGD resulting from surface fault rupture, landslides and soil liquefaction related phenomena including vertical settlement, lateral spreading, differential settlement and buoyancy movement

For WWSP project, majority of the pipeline is planned to be steel pipe with welded joins. In general, strong shaking generated transient stress and strain are not significant issues for welded steel pipe due to its high strength and flexibility. However, PGD hazards are considered the main concern for steel pipe performance during and after the earthquake.

For WWSP, risk for fault rupture is considered low and seismic landslide hazard is limited to a few steep localized areas along the corridors. Soil liquefaction is caused by the drastic increase of pore-water pressure (excited by the rapid cyclic earthquake ground shaking) and dramatic decrease of effective contact stress between soil particles which leads to the substantial loss of shear strength of the soil matrix.

EXPECTED TYPE AND RANGE OF SOIL LIQUEFACTION PHENOMENA

PGDs manifested by soil liquefaction mainly include post-liquefaction settlement, lateral spreading, flow failure and flotation. Post-liquefaction settlement is caused by the reconsolidation of liquefied soils during dissipation of the elevated pore water pressure, typically after the earthquake shaking stops. The reconsolidation typically leads to vertical settlement in the range of a few inches to more than 1 foot.

Lateral spreading is a form of ground failure, that is typically shown as blocks of mostly intact surficial soil crust on top of the liquefied layer moving down slope on very gentle inclinations (in some cases less than 2 degrees), or towards a free-face, such as a river channel or bluff. It is caused by the dramatic reduction of shear strength when soil liquefies, which leads to ground instability under the combination of seismic cyclic loading and static gravity loading. In past large earthquakes, lateral spreading typically occurred within 1,000 feet from the river banks and has generated large horizontal PGDs ranging from a couple of feet to more than 10 feet.

Flow failure is also caused by the substantial loss of shear strength of the liquefied soils. However, flow failure has typically occurred at steeper slope areas (Youd 1978) near rivers or at the steeper river banks where even the static stability of the soil mass cannot be maintained by the residual shear strength of the liquefied soil (Kramer 1996). In other words, the flow failure is driven by the static gravity stress, and the seismic cyclic stress only “triggers” the soil liquefaction and the associated strength loss which bring the soil to an unstable state. The flow failure is typically characterized with sudden failure, rapid flow movement, and large movement distance (sometimes exceeding tens of meters).

Soil liquefaction creates a buoyancy effect upon buried structures or pipelines. Numerous flotation cases of underground tanks, structures, manholes and pipelines have been reported in previous earthquakes, with upward movements ranging from a few inches to a few feet.

For the WWSP, post-liquefaction settlement and lateral spreading movement are considered as the main liquefaction manifested PGDs. Various degrees of liquefaction settlements will likely occur in the liquefaction hazard areas identified in Figure 4, and lateral spreading will likely affect the areas a few hundred feet on both sides of Tualatin River and its major tributaries. Flow failures may also be possible at some locations, but will likely be concentrated in some small areas along steep river and creek banks. Additionally, risk for floatation of the pipeline is considered to be low because the general pipeline depths are with or near the non-liquefiable soil crust (discussed above).

HISTORIC PIPE FAILURE LOCATIONS WITHIN LIQUEFACTION ZONE

Pipelines within lateral spreading and flow failure zones suffered severe damages during the past earthquakes (Eckel 1967, O’Rourke and Tawfik 1983, and O’Rourke et al. 1989). The authors have noticed that remarkably high damage ratios of pipelines were located at liquefied ground near the boundaries between the liquefied and non-liquefied area and concluded that the damages were due to sharp change of ground characteristics at soil boundaries.

For continuous pipelines depending on the pipe alignment relative to the ground failure direction, large axial tension/compressive stresses, and bending/shear stresses will likely developed at the boundaries of the lateral spreading/flow failure and between major ground moving blocks (O'Rourke and Lie, 2012). Additionally, flotation also caused pipeline failure within liquefaction zones. But this failure mode was observed more concentrated to sewer pipes (ALA 2005).

EXPECTED PIPE FAILURE LOCATIONS IN WWSP

Considering the relatively high liquefaction hazard within WWSP areas, the potential pipeline failures will likely occur at large liquefaction PGD areas including:

- Boundary zones between liquefiable soils and non-liquefiable soils,
- Within lateral spreading and flow failure zones,

TYPICAL PIPE FAILING MECHANISMS

Strong ground shaking and ground deformations may act in different directions related to the pipeline alignment and may be affected by axial and compressive forces or large bending moments caused by soil and pipe interaction that may locally damage steel water lines, or cause their failure. The four, most expected failure modes for continuous welded pipelines include:

- **Pipe fracture due to extensive tensile strain** related to pipe wall structure and tensile capacity of lap joints which are usually the weakest points.
- **Pipe wall local buckling or wrinkling due to extensive compressive strain** influenced by D/t ratio, the presence of internal and external pressure, and on the yield stress of steel material. If the ductility of the steel is not exceeded, buckled pipeline will be able to fulfill the basic function of carrying the flow.
- **Beam buckling due to extensive compressive, axial loading**, insufficient cover depth and lateral resistance of surrounding soils. In shallow trenches and loose backfill this mode of failure may occur. In deep trenches with and dense backfill, the pipe will develop local buckling before beam buckling.
- **Pipeline welded slip joints failure** (fracture or crushing).

MITIGATION MEASURES AGAINST SEISMIC ACTIONS AND ABILITY TO RESIST LARGE PGD TO MEET RESILIENCY REQUIREMNTS

Following the 9.0 Cascadia earthquake, it is expected that many local roads and bridges may not survive. Availability of spare parts (pipes, butt straps, valves, etc.) due to large pipe diameter, and their transport from different states to the pipe damaged areas may significantly exceed the time to meet the transmission facilities operational schedule suggested by the 2013 Oregon Resiliency code. Silty and sandy soils may liquefy, and access to pipeline alignments outside major roads may not be possible. Other utilities located within the same corridors may also break and require simultaneous repair action. Availability of heavy construction equipment could be questionable and its transport to potential pipe damaged locations may take significant time. Availability of shoring for deep excavation to access the pipe and availability of specialized labor forces to do repair work could also be problematic.

Designing the pipeline that may not break is almost impossible and/or cost prohibitive, as one cannot predict exact earthquake magnitude and duration, ground shaking intensity and direction, site specific PGD and differential settlement/movements. For example, several oil and gas pipelines with relatively thick wall (low diameter "D" to wall thickness "t" ratio, (D/t < 80) compared to water lines (D/t >100) with butt welded joints were damaged during major earthquakes in the past. (Rourke, Palmer, 1996)

On the other hand, City of Hillsboro and TVWD shall further explore geotechnical conditions along selected pipeline segments to be able to evaluate potential mitigation options and approaches to improve design and resiliency of transmission facilities, minimize construction and post-earthquake repair cost, and explore means and methods to repair and bring transmission facilities to expected operational conditions to meet 2013 Oregon Resiliency Requirements. In selecting potential mitigation options, the following important issues and options should be considered:

A) Soil Conditions along Pipe Alignments

- Provide more detailed geotechnical investigation along the pre-selected pipe corridors to identify major liquefaction concerns and hazard related areas.
- Identify locations and depths of liquefiable soil layers and estimate potential, amount and directions for liquefaction induced lateral spreading, and potential for flotation or settlement.
- Check if the pipe would be located within, above or below liquefiable soil layers.
- Identify locations of expected changes from liquefiable to non-liquefiable soils along pipe alignments, measure distances and widths of those layers to estimate seismic loads and stresses within liquefiable and non-liquefiable soil layers.
- Consider selecting routes outside liquefiable zones, or re-route pipeline alignment to avoid majority of liquefaction areas, if possible.

B) Pipe - Soil Interaction

- The smaller the pipe frictional forces on the pipe (pipe-soil interaction), the greater the capacity of the pipe for surviving soil movement,
- Provide adequate pipe trench cover to avoid pipe flotation, and beam buckling,
- Consider placing pipes in a trench with shallow sloping sides to be able to accommodate itself to the transverse as well to longitudinal components of the soil movement in part by moving slightly out of trench.
- Improve ground conditions or use deep foundation piles under the pipe to reduce amount of ground settlement, in areas expecting high subsidence.
- Consider using trenchless applications to install the pipeline below and beyond the liquefaction zones.
- In soil transition areas, stiff soil conditions introduce higher stress and strains in the pipeline. Use of soft backfill soils would result in reduced stresses but it also may reduce its resistance in global buckling.

- For high expected pipe compression/buckling or bending areas, plan for access to those areas following the major earthquake, due to high potential for damage.
- Consider providing in line valves in non-liquefiable soils, at adequate distance upstream and downstream of major hazard zones and sufficient size pipe/valve outlets to allow for installation of temporary bypasses, if needed

C) Pipe Characteristics

Steel pipelines strength to withstand different external forces are based on pipe diameter, pipe wall thickness, and d/t ratio, knowing that:

- The lower the grade of steel, the more yielding could take place, and the greater is the capability to resist liquefaction phenomena. Material ductility and deformation capacity may be more important than strength for pipe to survive operational capabilities after the earthquake
- Pipe movement capacity increases slightly with increasing pipe wall thickness.
- Increase in wall thickness will increase pipeline strength against seismic actions.

D) Pipe Joints

Pipes are connected using different pipe joints including full penetration butt weld and single and double lap welded joints, and flanges.

Full Penetration Butt Welded Joints

- Provide joint efficiency between 95% and 100 % of the pipe.
- Mostly used in high seismic areas and for thicker oil and gas pipelines, to withstand higher pressure and provide higher confidence and safety levels.
- Have higher installation cost.

Single Lap Welded (bell and spigot) Joints

- Have joint efficiency of approximately 40% to 45%.
- The strength of single welded joints are influenced by pipe wall thickness, pipe bell geometry, alignment of pipe bell and spigot, uniformity of eccentricity between spigot outside and pipe bell inside diameters.
- Are used in water mains due to lower internal pressure and lower construction cost. The aligning of pipes (fit-up) is easier and take less time to weld compared to butt welds.
- Could be welded internally for pipes larger than 30-inches or externally. Efficiency of inner weld slip joints are often larger than outer welds as the eccentricity of the weld in respect to pipe radius of inner welds are smaller than for outer weld. However, some tests showed opposite results.
- Lap welded joints are geometrically eccentric that introduces additional stress within the joint. As a result of this eccentricity, several case studies have documented the poor performance of single lap welded joint in earthquakes, which raises concerns about their use in regions of high seismicity, as described by O'Rourke and Liu (1999).
- Bell radius should be large enough so that an excessive amount of steel stability is not used in forming the bell.

Double Lap Welded (bell and spigot) Joints.

- No significant changes in joint efficiency between single and double welded joints because of failure mechanism (joint efficiency of 45 % for single and 55% for double welded joints)
- New testing results for double welded pipe joints, expected in the near future

E) Emergency Repair Planning and Readiness

As mentioned previously, designing the pipeline that may not break is almost impossible. Therefore emergency response planning shall be developed to address the necessary repairs and/or emergency bypass. This involves emergency response procedures, post-disaster access planning, emergency condition and damage assessment, establishing repair priority and strategy, and personnel and logistic readiness. It also involves storage at appropriate places of sufficient lengths of bypass piping, valving, apparatus for welding or joining pipe sections, fuel, and education of staff to be familiar with the equipment and emergency response planning.

SUMMARY

Seismic hazards and their potential effects on life lines have been recognized and addressed in the 2013 ORP by the State of Oregon. As a local critical water transmission pipeline, TVWD and the City of Hillsboro have established the design goal of seismic resiliency for their WWSS project. The main seismic event and hazard effect are considered to be the large magnitude CSZ earthquake and the associated soil liquefaction PGDs. Based on historical pipeline failures within liquefaction zone and failure mechanisms, the design focuses of the planning and routing levels of the WWSS are:

- Sub-route selection to minimize pipeline exposures to liquefaction PGDs,
- Investigation of pipe-soil interaction and soil improvements,
- Selection of appropriate grade and section for the steel pipe,
- Selection of appropriate welded joints,
- Emergency repair planning and readiness.

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Addressing Rehabilitation Challenges for the Underwood Creek Force Main

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Abstract

This paper discusses the challenges associated with the rehabilitation design and construction of the 36-inch ductile iron pipe (DIP) Underwood Creek force main. Based on the condition assessment and risk analysis, the Milwaukee Metropolitan Sewerage District (MMSD) minimized the risk of continued pipe failures by rehabilitating the force main. The rehabilitation design was based on a review of the currently available technologies and methods to identify those that appeared to be feasible. The four methods selected for further evaluation were cured-in-place pipe (CIPP) (for pressure), swagelining, tight liner (rolldown), and fold and form rehabilitation technologies. The final design documents included the use of two methods, CIPP and high density polyethylene (HDPE) liner, allowing the contractor to select the method and provide MMSD with the most cost-effective solution. The challenges that were addressed included minimizing the number of pits for construction of the liner because of location and constraints, the need to minimize impacts on the Underwood Creek Parkway and crossing a railroad and highway. This paper also discusses the challenges associated with implementing the design during construction and the coordination with many stakeholders, including the full reconstruction of a U.S. Highway 45 interchange by the Wisconsin Department of Transportation (WisDOT), and the construction of overhead high voltage transmission lines by the American Transmission Company (ATC).

PROJECT BACKGROUND

The Milwaukee Metropolitan Sewerage District's (MMSD's) Underwood Creek pump station and force main were designed in 1981, and construction was completed in 1983 to provide an additional 34 million gallons per day (mgd) of relief flow capacity to the Underwood Creek Metropolitan Interceptor Sewer System during peak flows. The 36-inch ductile iron pipe (DIP) force main is approximately 10,000 feet long and runs from the Underwood Creek pump station to a point on Watertown

Plank Road where it transitions to gravity flow as shown on Figure 1. The force main alignment runs along Underwood Creek for approximately 5,800 feet and then runs in the center of Watertown Plank Road for the remaining 4,200 feet. The pipe is CL50 ductile iron with a cement mortar liner and an asphaltic coating. The force main was installed with an 8 mil polywrap and sand bedding. The force main was constructed with bonded joints to be electrically continuous, and 11 test stations were installed so that the electrical potential between the force main and the ground could be monitored.

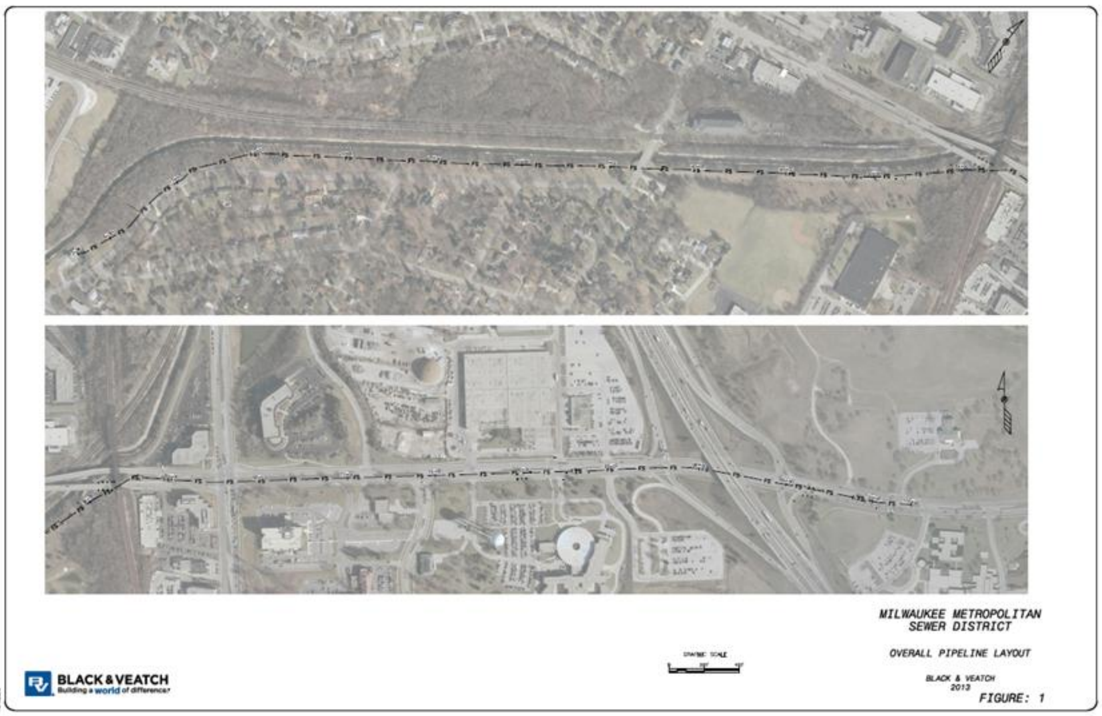

Figure 1. Underwood Creek force main alignment. (Source: Black & Veatch)

The force main experienced two leaks along Underwood Creek, one in October 2007 and one in June 2009. Following each incident, the MMSD initiated investigations into the causes for the failures. These investigations concluded that the likely cause of failure was localized external corrosion of the pipeline, because of a damaged or improperly installed polywrap, exposing the force main to corrosive soils.

In 2009, following the second leak, a consultant helped the MMSD develop an approach for inspection of the force main to determine the existing condition of the pipe. The approach included specific inspection technologies in distinct phases using indirect condition assessment methods to identify areas that were the most corrosive, followed by direct condition assessment (pipe inspection and wall thickness measurements) with ultrasonic methods in the areas most susceptible to corrosion.

The condition assessment results were reviewed with MMSD and several alternatives were presented. The alternatives ranged from regular leak detection to rehabilitation of the pipeline. MMSD evaluated the alternatives and the associated risks and elected to install a new liner in the pipeline to minimize the risk of exposure from another failure of the pipeline. The next step was to complete a preliminary design to evaluate rehabilitation technologies to determine which methods should be considered.

PRELIMINARY PROJECT DESIGN PROCESS

The preliminary design was initiated following the condition assessment with evaluation of several alternatives. There are currently many technologies available for rehabilitation, and they were reviewed in the preliminary design. The spray-applied semi-structural liners were not considered because the previous failures appeared to be from external corrosion, and it was decided to look at structural rehabilitation, assuming there were large holes in the host pipe. The following methods were considered for evaluation:

- Sliplining.
- Pipe bursting.
- Cured-in-place pipe (CIPP).
- Swagelining and tight liner.
- Fold and form pipe.

Sliplining. Sliplining would significantly reduce the potential of leaks from further corrosion of the DIP since it creates a new pipe within the existing pipe. The use of sliplining was not considered because it significantly reduces the cross-sectional area of the flow by inserting a smaller diameter pipe into the existing pipe that would impact the overall system operation.

Pipe bursting. The use of pipe bursting could be performed for the required project diameter and would allow the use of high density polyethylene (HDPE) or fusible polyvinyl chloride (PVC) pipe. However, pipe bursting for this diameter has the potential for heaving in Watertown Plank Road and may be problematic in the relatively sandy soils that are present along Underwood Creek.

Because of the project-specific risks associated with pipe bursting, this technology was not considered for rehabilitating the Underwood Creek force main.

Cured-in-place pipe. CIPP for pressure applications is a newer technology gaining popularity in the rehabilitation of pressure pipe. There are several manufacturers of this product with similar application requirements.

The system can be designed as a fully structural, Class IV system in accordance with American Society for Testing and Materials (ASTM) F1216 guidelines. This system does not rely on the host pipe for any structural support. The system can be designed as a Class III semi-structural system, which relies on the host pipe for some structural support. A diagram of a CIPP retrofit is presented on Figure 2.

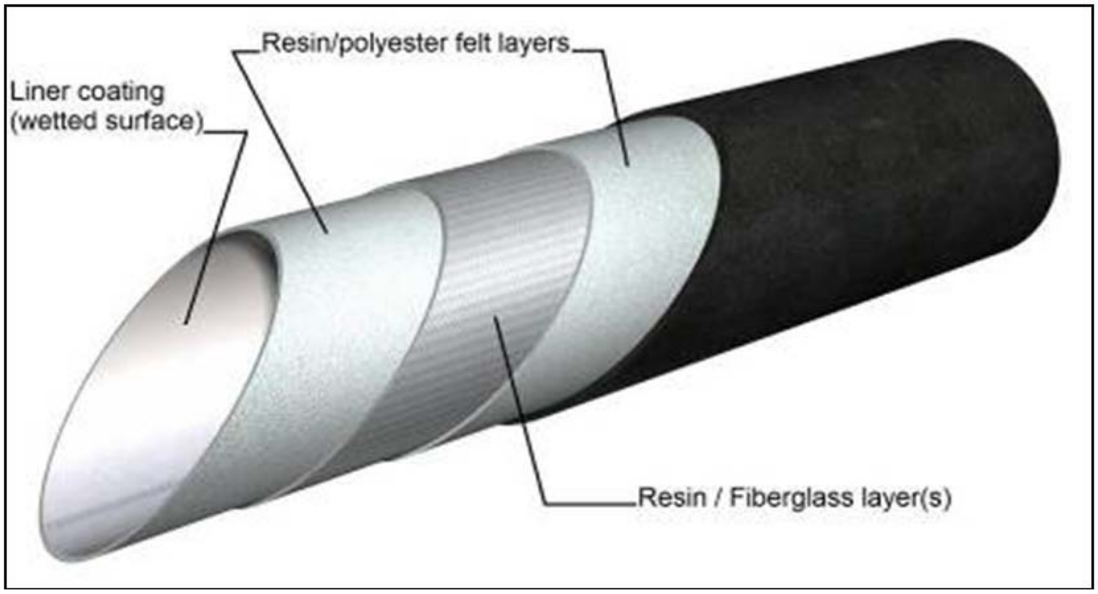

Figure 2. Cured-in-place pipe components. (Source: Norditube)

The installation of a cured-in-place liner would reduce the potential of leaks from further corrosion of the DIP. The hydraulics of the cured-in-place liner has an improved flow characteristic and an increase in flow capacity resulting from the improved friction factor. The final design would have to evaluate the impact of the liner on the operation of the pumps.

Design of CIPP is determined by the normal working pressures in the force main, and when serving as a structural replacement, the external loading conditions are included. The normal working pressures and loadings were confirmed during the design process. The use of CIPP was considered for the rehabilitation of the force main.

Swagelining. Swagelining is a trenchless rehabilitation process that involves running the inserted HDPE pipe through a die to slightly reduce its diameter and allow it to be pulled through the host pipe, as shown on Figure 3. Once through, the tension is relieved and the new lining will elastically return to its original dimensions. The process can utilize either thin or thick polyethylene material. The thickness of the liner is determined during the design process.

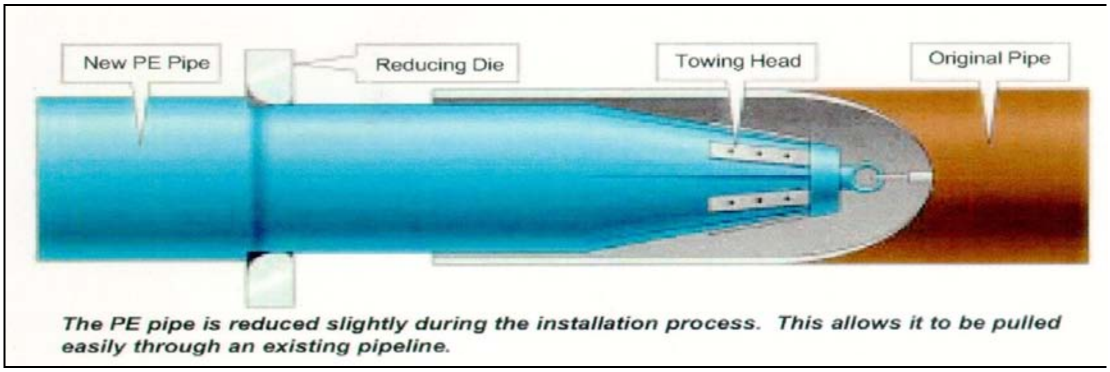

Figure 3. Swagelining process. (Source: Murphy Pipelines)

The hydraulics for polyethylene liners has an improved flow characteristic and an increase in flow capacity resulting from the improved friction factor. The final design would have to evaluate the impact of the liner on the operation of the pumps. The use of swagelining with HDPE was considered for the rehabilitation of the force main.

Fold and form. This installation process is similar to cured-in-place but uses a close-fitting polyethylene pipe that is custom designed to match the existing conditions of the pipe to be rehabilitated. The pipe is fused together in length for the project and then held together by bands that are broken after installation into the host pipe, as shown on Figure 4. The polyethylene liner will improve the flow characteristics in the force main and any flow loss from the reduced diameter is typically recovered. The fold and form process using HDPE was considered for the rehabilitation of the force main.

Figure 4. Fold and form process. (Source: Insituform)

Results of the preliminary design determined that the CIPP, swagelining, tight liner (rolldown), and fold and form lining should be further evaluated in the design of the rehabilitation.

Table 1 provides the design criteria that were established for the rehabilitation:

Table 1. Rehabilitation Design Criteria.

Design Parameter	Value
Host Pipe	36-inch DIP
Flow Capacity	34.0 mgd
Internal Working Pressure	52 psi
Occasional Surge Pressure (emergency shutdown)	120 psi
Soil Depth (above crown)	15 feet
Soil Density	110 lb/ft^3
Ground Water Depth (above crown)	3 feet
Live Load, HS20	0.0 psi (buried greater than 8 feet)
Modulus of Soil Reaction	1,000 psi
Modulus of Elasticity HDPE PE 4710 (50 year design)	28,500 lb/in.^{2}
Modulus of Elasticity CIPP	350,000 lb/in.^{2} (Norditube CIPP resin)
Soil Support Factor	1.0

With the potential rehabilitation methods identified, the next step was to compare each alternative to identify advantages and disadvantages for each. The evaluation included a review of the use of PE 4710 that, at this time, was not included in the standard. Based on the brief review for this application, it was determined that it provided the required strength to meet this design. Table 2 summarizes the design criteria for those methods considered for further evaluation.

Table 2. Design Criteria.

Technology	Pipe Pressure Rating	Surge Pressure Rating(1)	External Load	Liner Material	Liner Wall Thickness	Flow Capacity (2)	Technology Constraints	Distance Needed Between Pits	Time to Construct
CIPP	80 psi	160 psi (Design Safety Factor [SF] of 2)	ASTM F1216 Design	Felt w/glass fiber reinforcement Epoxy Resin	Approx. 0.75 inch (18 mils)	34.5 mgd	Limited to less than 45 degree angle; temperature impacts cure time	Approximately 500 to 700 feet	15-20 weeks
CIPP	83 psi	166 psi (Design SF of 2); short-term burst pressure of 325 psi	ASTM F1216 Design	Felt w/glass fiber reinforcement Epoxy Resin	Approx. 0.75 inch (20 mils)	34.5 mgd	Limited to less than 45 degree angle; temperature impacts cure time	Approximately 500 to 700 feet	15-20 weeks
Swagelining (Thick Pipe)	101 psi	202 psi	9.66 SF against buckling	HDPE – PE 4710 (3) DR-21	1.8 inches	33.0 mgd	Thicker wall limits bends, increases pull times; temperature impacts roll down	Longer along Parkway 2 – 3,000; Water-town Plank will meet conditions	6-8 weeks after pits are ready
Swagelining	81 psi	162 psi	6.91 SF against buckling	HDPE – PE 4710 (3) DR 26	1.4 inches	33.0 mgd	Thinner wall negotiate bends; temperature impacts roll down	Longer along Parkway 2 3,000; Water-town Plank will meet conditions	6-8 weeks installation after pits ready
Swagelining (Thin Pipe)	64 psi	128 psi	4.89 SF against buckling	HDPE – PE 4710 (3) DR 32.5	1.1 inches	33.5 mgd	Thinner wall negotiate bends; temperature impacts roll down	Varies with existing conditions	6-8 weeks installation after pits ready
Tight Liner (Rolldown)	64 psi	128 psi	4.89 SF against buckling	HDPE – PE 4710 (3) DR 32.5	1.1 inches	33.5 mgd	Negotiate bends < 45 degree	Estimated 1,500 feet, varies with conditions	6 weeks for installation after pits ready
Subline (Fold/Form)	64 psi	128 psi	4.89 SF against buckling	HDPE – PE 4710 (3) DR 32.5	1.1 inches	33.5 mgd	Negotiate bends < 45 degree	Lengths depending on conditions	6 weeks for installation after pits ready

Notes:
(1) Surge pressure allowances in AWWA C906 are applied above the pressure class, and for occasional surges, are equal to the pressure class.
(2) Flow capacity with four of the five pumps in operation. Existing DIP estimated capacity is 31.0 mgd based on C value of 110.
(3) PE 4710 is the newer polyethylene that is being used; the design factor allows for a higher pressure rating.

Other factors. The preliminary design also had to consider how the project would be impacted by other projects planned for the area.

There are two proposed projects that would impact the rehabilitation of the Underwood Creek force main. The first is a proposed plan by the Wisconsin Department of Transportation (WisDOT) to revise the intersection of Highway 45 and Watertown Plank Road as part of the Interstate 94 Zoo Interchange Project. A preliminary site layout in the vicinity of Watertown Plank Road is presented on Figure 5. As shown, Watertown Plank Road is scheduled to be reconstructed along the entire length of the Underwood Creek force main. The modifications will increase or decrease the depth of cover one to two feet in the area from Manhole No. 34704 (near Watertown Plank Road and the railroad bridge) to Manhole No. 34909 (near the entrance to the County Parks Building). The changes in the final depth of cover, as well as the planned construction of new bridge piers by the force main, will require that WisDOT structurally assess the changed loading conditions on the pipe. However, the planned construction does not afford any appreciable cost-saving opportunity to either add bonding to the pipe or replace the pipe because the depth of cover over the pipe will not be significantly reduced.

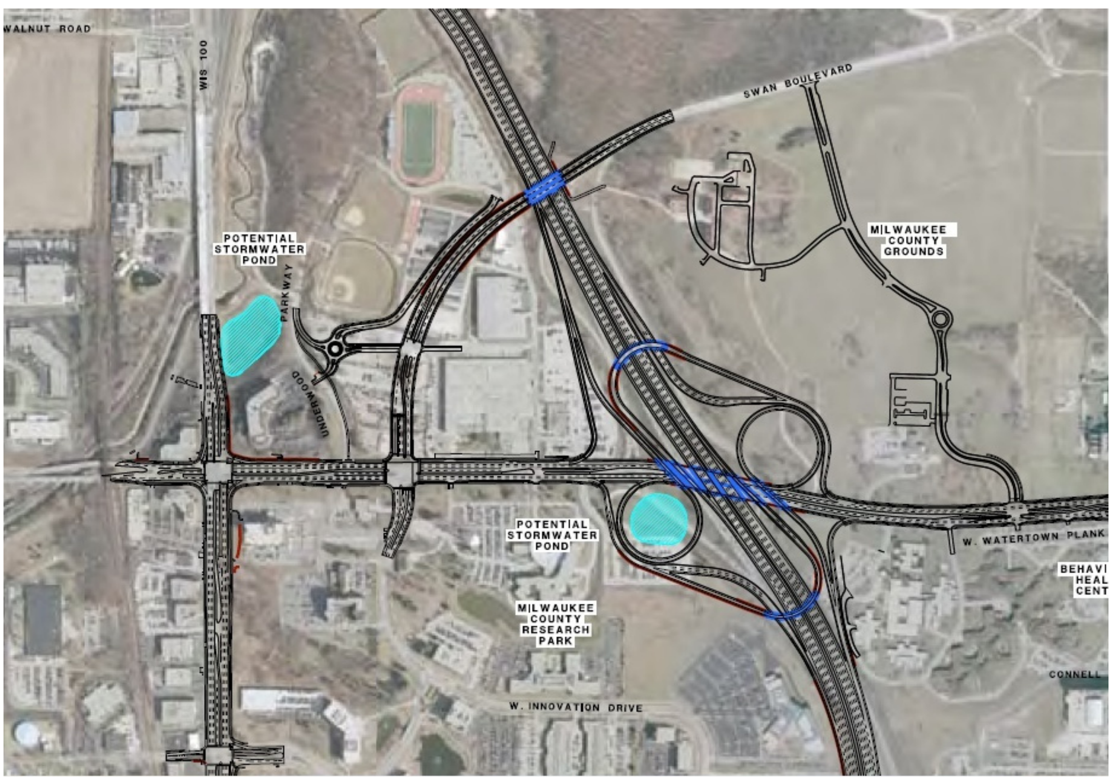

Figure 5. DOT planned changes to Watertown Plank Road. (Source: WisDOT)

The second project is the proposed Western Milwaukee County Electric Reliability Project to construct power transmission lines to a new substation north of Watertown Plank Road. There are several routes being considered. Route B would construct overhead power along the Underwood Creek Parkway near the force main and then construct underground power along Watertown Plank Road. At this point, the final route alignment is being determined by the Public Service Commission. If Route B is selected, American Transmission Company (ATC) would need to complete a more detailed design of the underground power line along Watertown Plank Road. After a review of the ATC design, MMSD could decide if the ATC project affords a better opportunity to re-bond or replace the force main.

Because of the potential for stray currents from high voltage power lines to induce corrosion on buried metallic pipe, ATC was required to demonstrate there would be no adverse impact on the force main from any potential stray currents from this power line.

The impact from these two projects was included in the final design.

FINAL PROJECT DESIGN PROCESS

The final design process began by refining the methods that could be constructed with the limitations imposed by the existing alignment and surface conditions, the layout of the project site and avoidance of other work in the area. Primary constraints considered for the final rehabilitation method selection included the requirements for accessing the pipeline via excavation pits (including material layout lengths), pipeline geometry and constructability challenges to meet operating conditions and parameters.

The existing alignment of the pipeline included both horizontal and vertical changes in direction, including some bends greater than 11-1/4 degrees. The alignment also was located in environmentally sensitive regions, an active park and bike path, beneath a major roadway and under a freeway crossing – all conditions affecting potential excavation pit locations and pipeline layout lengths. These existing conditions required the selected methods to be achievable while meeting the requirements and limitations of the various stakeholders, including a large portion of the public.

The critical nature of the force main during a rainfall event also resulted in the necessity of its placement back into service within a three-day period, should the long-term forecast predict significant rainfall. Any bypassing of the force main was limited because of this constraint, thus the rehabilitation methods selected had to meet these requirements so that full reliability would be ensured within a short duration.

These constraints had a significant impact on the selection of the final rehabilitation method. Early in the final design, CIPP and swagelining with HDPE were selected as the final rehabilitation methods. The next step in final design was to determine the locations in which either or both technologies were feasible. The alignment was separated into four segments based on the project constraints outlined above. Segments A through D are presented on Figure 6.

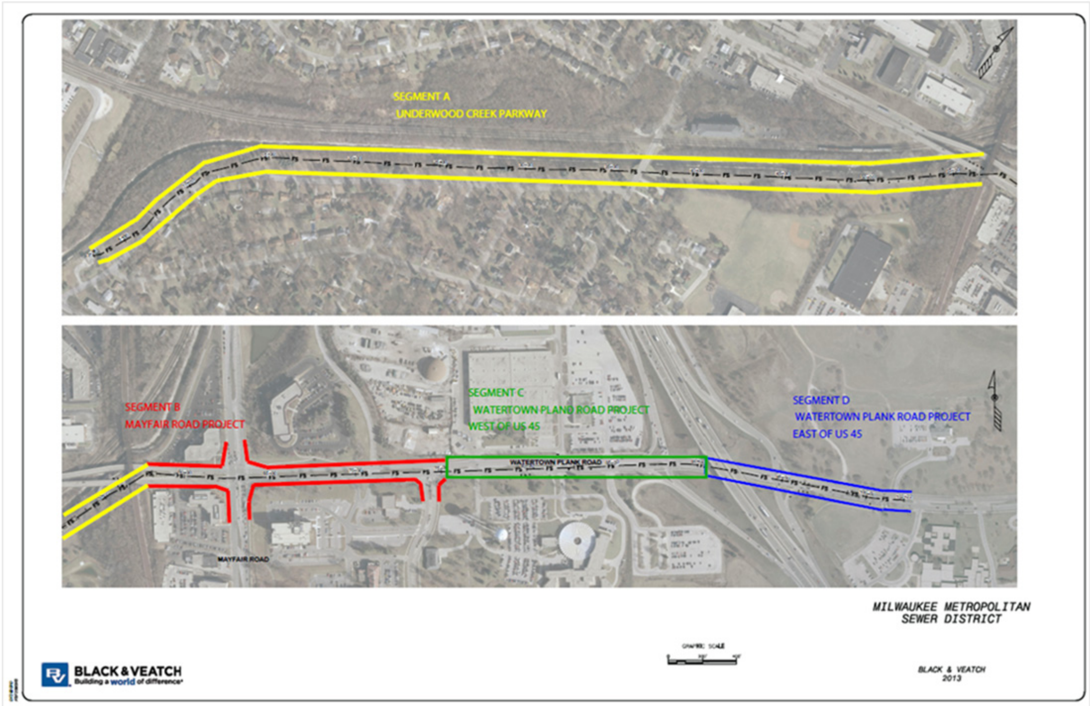

Figure 6. Project segments. (Source: Black & Veatch)

Segment A: Both swagelining and CIPP rehabilitation techniques are applicable along Segment A. Swagelining the first several hundred feet of this section would require several pits at existing horizontal bends. Alternatively, the swagelining contractor could elect to use CIPP in this first section to avoid digging multiple access pits. The remainder of the segment is well suited for swagelining because of ease of access and long straight reaches. Given the large amounts of staging areas available in Segment A, MMSD may benefit from competitively bidding the two technologies against each other, even if the swagelining contractor needs to use a CIPP liner for the first several hundred feet.

Segment B: Work associated with rehabilitation of Segment B will be performed by coordinating with the reconstruction of a portion of Watertown Plank Road in the vicinity of Mayfair Road. Use of CIPP rehabilitation technology for this segment is recommended.

Segments C and D: The rehabilitation of the force main in these segments will be designed as CIPP because of the limitations imposed by construction from the WisDOT project and active roadway conditions.

A challenge during final design was the selection of the excavation pit sites and access to the sites. Because of the environmentally sensitive locations along Underwood Creek and the necessity to maintain traffic flow on Watertown Plank Road, there were limited site locations. The design team overcame these constraints by working with stakeholders and utilizing the correct rehabilitation method to limit impacts on the environment and community.

A key step during final design was development of specifications to address the planning of the project installation and the confirmation of a quality product installation. These included development of detailed sections for pre- and post-installation closed circuit televising of the pipeline, cleaning of the existing force main prior to installation, and bypassing requirements and limitations. The careful development of these specifications was implemented during construction when an unknown obstruction was discovered in the pipeline that would have resulted in detrimental impacts that would have compromised the final pipeline liner.

BIDDING AND CONSTRUCTION

The bid allowed the alternative for both rehabilitation methods. Based on the bids received, the preferred rehabilitation method was CIPP liner for the entire alignment, primarily because of the project conditions and field constraints, including access pit locations.

Construction of the pipeline began in late summer 2014 and continued through the winter. The number of access pits required several connections between the CIPP liner sections. The design required the use of PVC pipe to connect the CIPP liner at the access pit as shown on Figure 7.

36" PVC pipe, used to combine sections of lined pipe.

Figure 7. PVC used to combine sections of the pipe. (Source: MMSD)

The challenge of construction was increased with the cold weather and required a large boiler system to create the steam to cure the resin. The CIPP process was successful, and the pipeline was rehabilitated.

CONCLUSION

The rehabilitation of the force main required the evaluation of several technologies to address the many challenges and limitations from the location of the alignment. The process from preliminary design through final design and construction included addressing these constraints to provide the liner that would extend the service life of the pipeline and minimize the risk of failure. The CIPP process was successful (as shown on Figure 8), and the pipeline was rehabilitated.

Figure 8. CIPP lining in pipe. (Source: MMSD)

Decision-Making Guidance for Culvert Rehabilitation and Replacement Using Trenchless Techniques

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Abstract

Culverts which are typically located under roadways and embankments for the passage of water are designed to support the super-imposed earth and live loads from passenger vehicle and trucks as well as the internal hydraulic loading from the stream flow. Many of the existing culverts in the U.S. are in a deteriorated state having reached the end of their useful design life, making them vulnerable to failures with potentially catastrophic consequences. Traditionally, deteriorated culverts have been replaced by the conventional open-cut construction method. Due to higher costs, adverse environmental and societal impacts associated with open-cut method, particularly in high population and busy roadways, transportation agencies are increasingly looking to adopt trenchless techniques for addressing their culvert problems. This paper reviews several trenchless rehabilitation and replacement techniques investigating their suitability to address different defects, and their compatibility with various host pipe materials and diameters. With focus on reinforced concrete pipe (RCP), corrugated metal pipe (CMP) and high density polyethylene (HDPE) culvert materials, easy-to-use decision-making flowcharts are presented in this paper. State transportation agencies, U.S. Forestry Service and other local government agencies that manage culvert infrastructure will benefit from this paper.

1. INTRODUCTION

The U.S. culvert infrastructures have served the American society for over 100 years (Selvakumar et al., 2014). Due to their invisibility from the surface, they often get ignored until a problem such as road settlement or flooding arises. Many culvert structures are currently in a deteriorated condition reaching the end of their design life (Yang et al., 2009). Their state of disrepair is mainly attributed to: 1) a general lack of uniformity and improvement in design, construction, and operation practices; 2) insufficient quality control during pipe installation; 3) little or no inspection and maintenance; and 4) reduced funding (Ge et al., 2014). The deteriorated culvert infrastructure could potentially lead to surface depression, extensive cracking, and in extreme cases a collapse of the road surface.

Traditionally, deteriorated culverts have been replaced using the conventional open-cut construction method. Due to higher costs, adverse environmental and societal impacts associated with the open-cut method, particularly in areas of high population and busy roadways, transportation agencies are increasingly looking to adopt trenchless techniques for addressing their culvert problems. Therefore, several techniques for rehabilitating and replacing culverts have been developed thus far; however, some still need further validation for wider adoption in practice.

Previous studies have proposed decision-making frameworks for choosing culvert rehabilitation and replacement techniques. Thornton et al. (2005) presented a Multi-Criteria Decision Analysis (MCDA) tool in Microsoft Excel for selecting one appropriate culvert rehabilitation method from slip lining, close-fit lining, spirally wound lining, cured-in-place pipe lining (CIPP), and spray-on lining. Their decision-making is based on scores given to each rehabilitation technique considering several criteria such as design life, capacity reduction, installation time, etc. Matthews et al. (2012) developed a set of decision-making flowcharts for appropriately selecting a rehabilitation or replacement method for corrugated metal pipes (CMP). Their decision-making approach was based on specific defects normally observed in CMP culverts; namely, inadequate hydraulic capacity, inadequate structural capacity, and inadequate bedding support. Hollingshead et al. (2009) briefly summarized the description, installation procedures, and highlighted the advantages and disadvantages of segmental lining, spiral lining, CIPP, fold and form lining, deformed-reformed HDPE lining, and cement mortar spray lining in order to appropriately select a suitable rehabilitation method.

Although many researchers evaluated culvert rehabilitation and replacement methods, few have presented a decision-making framework that is classified by defects for individual culvert materials for better utility. This paper briefly reviews several trenchless rehabilitation and replacement techniques, highlighting their suitability to different defects and materials. With focus on reinforced concrete pipe (RCP), CMP and high density polyethylene (HDPE) materials, easy-to-use decision-making flowcharts for selecting appropriate culvert rehabilitation and replacement techniques for different defects are developed and presented in this paper. Some techniques were excluded in the analysis presented in this paper because of their limited use and lack of sufficient performance data.

2. TRENCHLESS TECHNIQUES FOR CULVERT REHABILITATION AND REPLACEMENT

This paper evaluates slip lining, cured in place pipe (CIPP), cement mortar lining, fold and form lining, spiral-wound lining, and pipe bursting techniques. Based on the evaluation of these techniques, easy-to-use decision making guidance is presented. This section briefly describes the aforementioned techniques with their specific advantages and limitations highlighted in Table-1 (Caltrans, 2013; Mitchell et al., 2005; Meegoda et al., 2009; Hunt et al., 2010; Hollingshead et al., 2009; Syachrani et al., 2010; and Yazdekhasti et al., 2014).

2.1 Slip lining (SL)

Slip lining entails inserting a smaller diameter pipe directly into a deteriorated or failed host culvert. It is accomplished by either pulling or pushing the liner pipe into the host culvert using jacks or other equivalent equipment as shown in Figure-1a. The space between the host pipe and liner is grouted forming a composite pipe that is stronger and smoother. Flexible pipes such as PE, HDPE or PVC, with mechanical (segmental) or fused (continuous) joints are typically used as liners.

2.2 Cured in place pipe (CIPP)

CIPP entails inserting a polymer fiber tube or hose impregnated (or coated) with a thermosetting resin (such as unsaturated polyester, epoxy vinyl ester, or epoxy with catalysts) into the host culvert through an inversion process, as shown in Figure-1b. The liner is then expanded to closely fit the host pipe after which it is cured using hot water, steam or UV light. CIPP can be applied to all shapes of host pipes, and the liners in this method are known for their flexibility and suitability for even 90 degree bends, making them ideal for cases where access to culvert is limited.

2.3 Fold and form lining (FFL)

Fold and form lining entails inserting a folded liner pipe into the host culvert after which the liner is heated and expanded to tightly fit into the host culvert. The liner is then cooled to maintain its shape. Figure-1c illustrates the fold and form lining method.

2.4 Spiral-wound liner (SWL)

Spiral-wound lining entails feeding coiled inter-locking plastic strips through a winding machine that moves along the host culvert forming a smooth plastic pipe, as illustrated in Figure-1d. The space between the host culvert and the plastic liner is grouted to form a robust composite pipe. This technique is more suitable in cases with non-circular host culverts and strict access restrictions.

2.5 Cement mortar Lining (CML)

Cement mortar lining method entails spraying cement mortar on the interior of the host culvert using rotating head of air-powered machine. The rotating or conical trowels then provide smooth surface on the inside, as can be observed from Figure-1e. This technique is typically employed in the case of steel or iron culverts.

2.6 Pipe bursting (PB)

Pipe bursting entails forcing a larger size expansion head into the host culvert which fractures or splits it while pulling along a new pipe, as shown in Figure-1f. Pipe bursting is one of the popular trenchless replacement techniques which can also be used for upsizing the culvert subject to favorable soil conditions.

Based on the advantages, limitations, and descriptions presented in Table-1 and the preceding paragraphs, decision-making guidance is developed. The guidance is based upon specific defects often observed with RCP, CMP, and HDPE culverts.

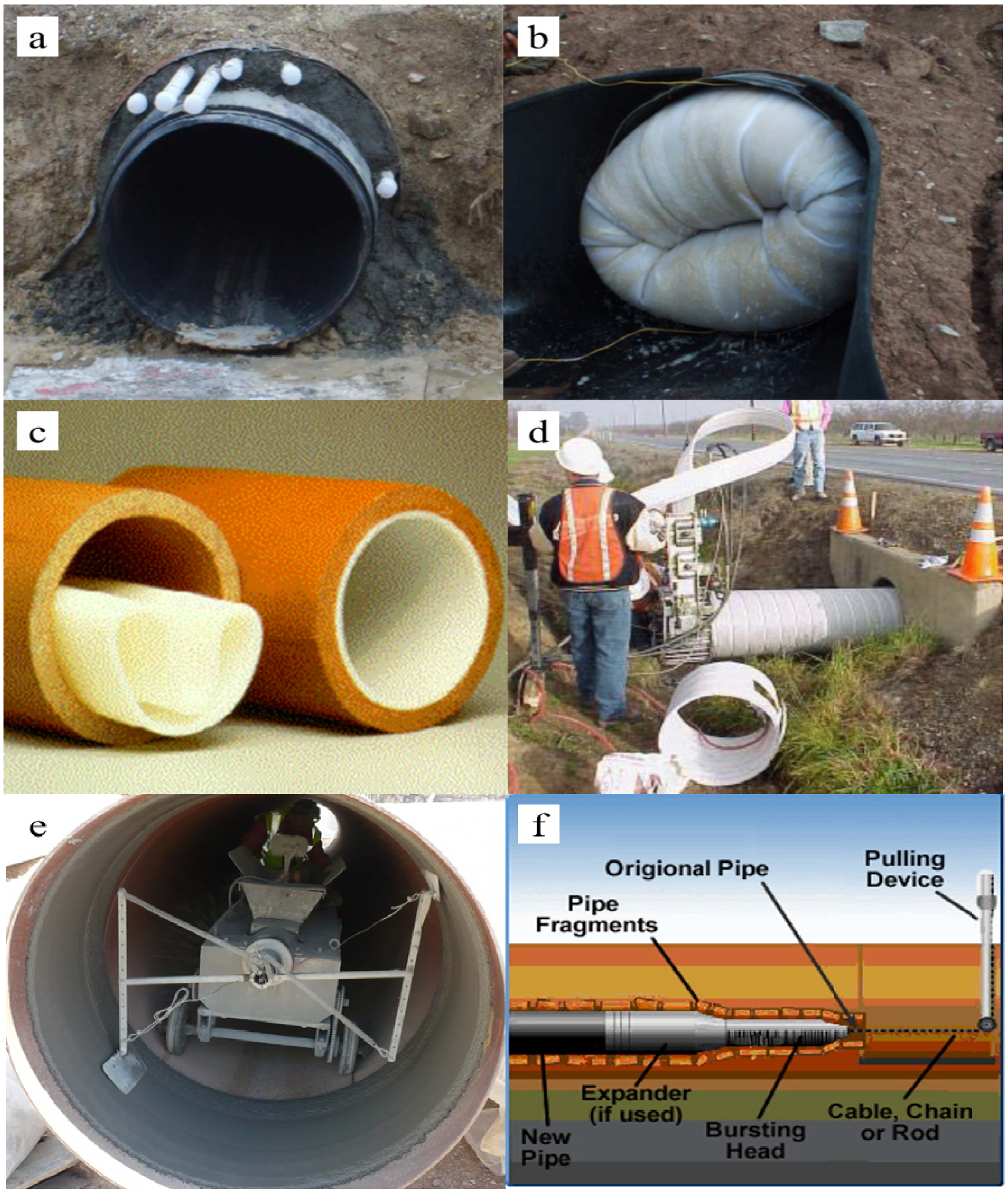

Figure 1. Illustration of rehabilitation and replacement methods: (a) Slip lining, (b) CIPP, (c) Fold and form lining, (d) Spiral-wound lining, (e) Cement mortar lining, (f) Pipe bursting

Table 1. Advantages and limitations of culvert rehabilitation and replacement methods

Technique	Advantages	Limitations
SL	• Simplest and cheapest technique • Can be used with live flow in host culvert • Offers structural capacity	• Reduced culvert size • Needs larger pits for liner insertion • Not easy to reconnect laterals
CIPP	• Requires no access pits • Can negotiate bends • Applicable for different culvert shapes and tight curves	• Need a lot of water or steam • Toxic resins could infiltrate ground water • Cannot be used with live flow
FFL	• Increased liner size compared to SL • Can negotiate bends • Doesn't need grouting	• Applicable to limited host culvert sizes and shapes • Toxic resins could infiltrate ground water • Requires additional resources for folding the pipe
SWL	• Can be used with live flow in host culvert • Applicable for different culvert shapes and tight curves • Requires no access pits	• Larger installations require man entry • Need specialized equipment • Reduced culvert size
CML	• Effectively fights corrosion in unlined metal culverts • Long-term protection at cheaper cost • Easy mixing and application of lining material	• Suitable for bends up to 45 degree • Requires significant curing time • Used mostly for corrosion protection only
PB	• Provide structure support • Capable of installing larger than host culvert size • Faster and cheaper than open-cut method usually	• Could pose threat to surrounding sub-structures • Not suitable for all soil conditions • Cannot fix line and grade problems of host culverts

3. DECISION-MAKING GUIDANCE FOR TRENCHLESS CULVERT REHABILITATION AND REPLACEMENT TECHNIQUES

The three common types of deficiencies that would constitute rehabilitation or replacement are (1) structural, (2) hydraulic capacity, and (3) bedding. Structural issues can include: collapse, corrosion/abrasion, invert deterioration, local damage in the wall or joint such as crack or spalls, shape distortion, or defective/misaligned joints. Hydraulic capacity issues can include roadway overtopping/flooding, scour at the inlet or outlet, sediment/debris buildup, and embankment damage. Most hydraulic issues are addressed with methods and practices that are outside of the focus of this paper such as culvert realignment, endwalls and wingwalls, energy dissipaters, and bank stabilization. However, in cases where additional capacity is needed (i.e., flooding), methods such as pipe bursting can be used to increase capacity. Bedding deficiencies can lead to surface depressions, voids around the culvert, and undermining defects. These issues can be addressed with various forms of grouting, which are outside of the focus of this paper. Rehabilitation or replacement techniques that are specifically suitable for commonly observed defects in RCP, CMP, and HDPE culverts are categorically summarized in the form of decision-making flow charts which are presented in Figures 2 and 3. These flow charts are developed based on the capabilities, advantages and limitations of the trenchless alternatives as discussed in SECTION-2, along with the support of the previously cited literature.

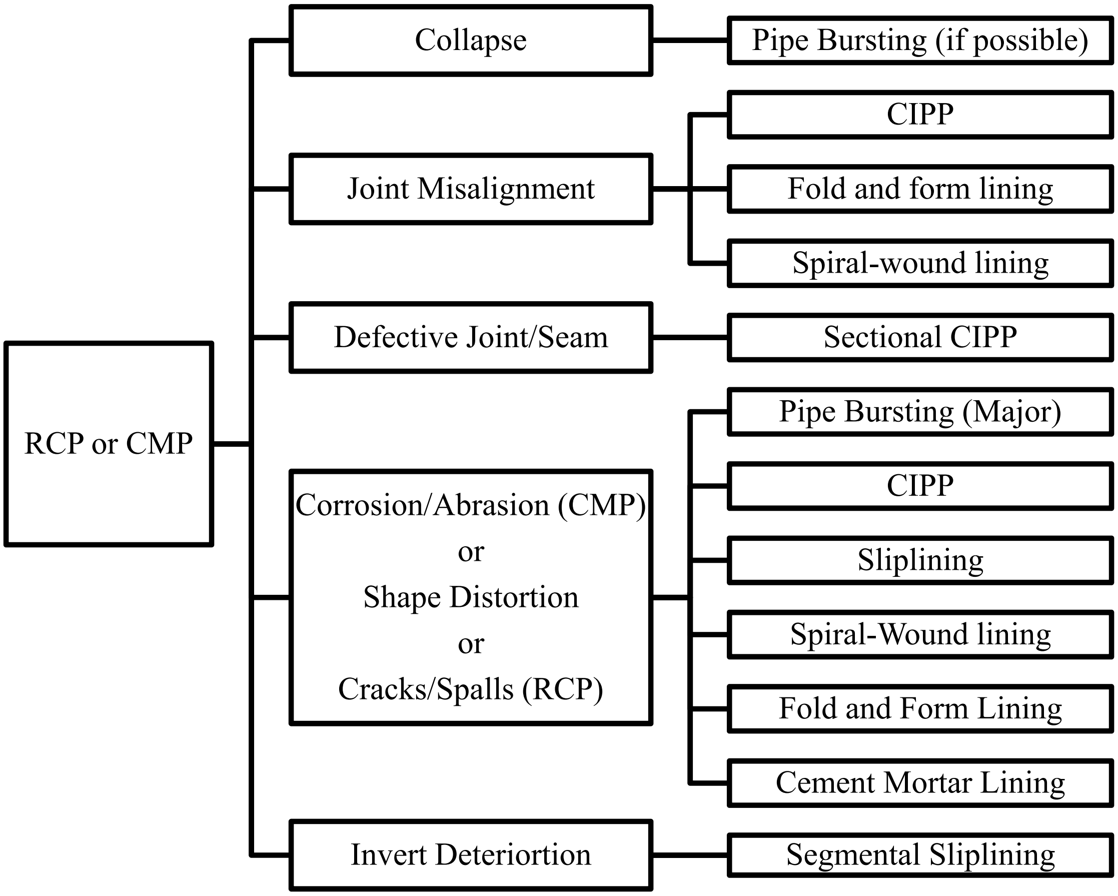

Figure 2. Trenchless rehabilitation and replacement techniques for CMP and RCP culverts

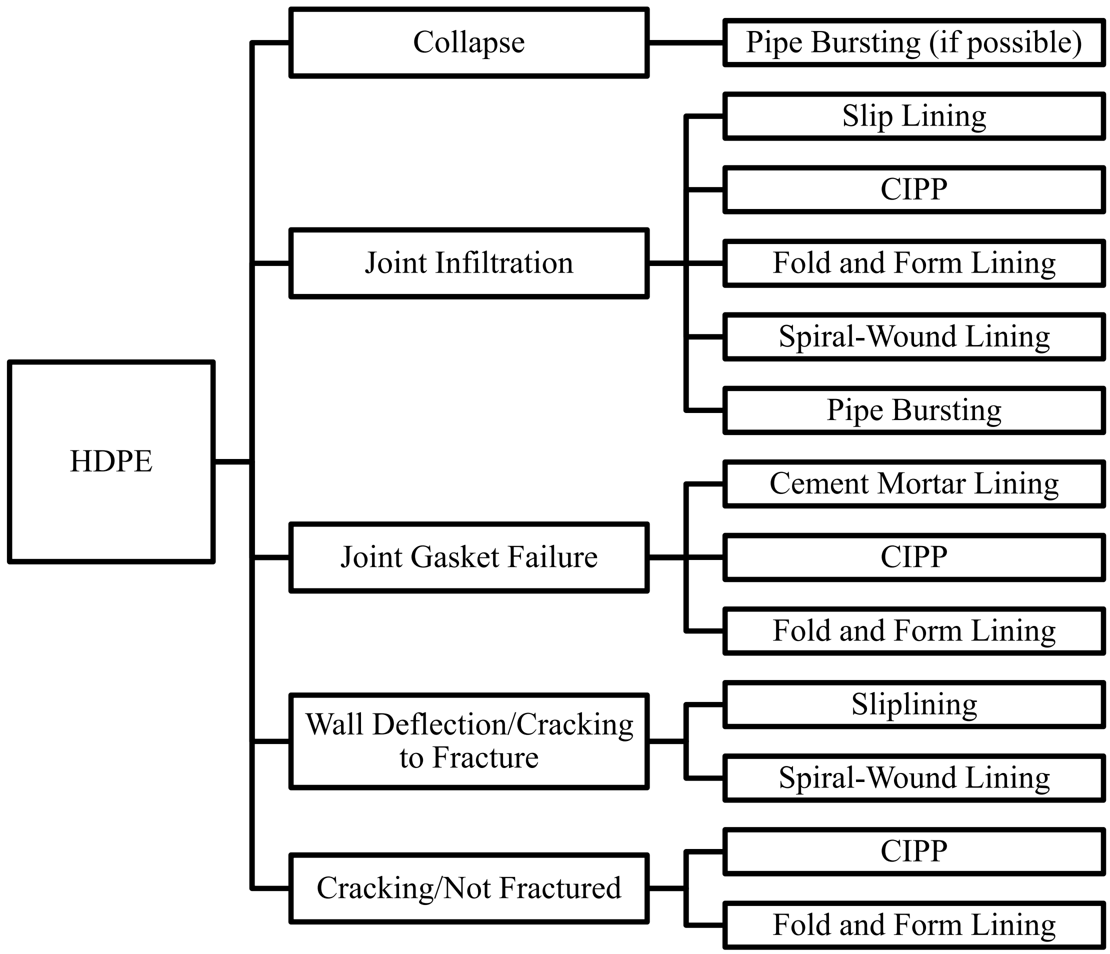

Figure 3. Trenchless rehabilitation and replacement techniques for HDPE culverts

4. CONCLUSION

This paper provides an overview of trenchless rehabilitation and replacement techniques that are applicable to commonly encountered structural issues in RCP, CMP, and HDPE culverts. The two flowcharts presented provide an easy to follow decision making process for when certain techniques are applicable. Additional considerations such as cost, contractor availability, and local preferences must always be taken into account before selecting a final technology, but these flowcharts provide a general process for selecting a group of technically applicable methods.

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Hot Tapping and Plugging Procedures Enable Replacement of Concrete Pressure Pipelines Reaching the End of Service Life without Service Interruption

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Abstract

As water pipelines and sewer force mains reach the end of their service life, leaks and uncontrolled line failures become more common. These failures may result in unacceptable environmental incidents and lack of adequate potable water and sewage service. This paper will discuss how hot tapping and plugging is used as an effective enabling procedure that allows the replacement or rerouting of concrete pressure water lines and sewer force mains while maintaining operations. The discussions in the paper will include: the tapping and plugging concept and sequence; engineering principals involved; forces developed in the application of the procedure and the need to address those forces with adequate thrust block designs. Two specific projects will serve as case studies to highlight the use of the technique. One case study is about the relocation and replacement of a prestressed concrete cylinder pipe sewer force main in an urban area. The other case study is the replacement of a municipal concrete pressure pipe (non-cylinder) water supply pipeline.

TAPPING AND PLUGGING ENABLES REPLACEMENT OF FAILING PIPE

As water pipelines and sewer force mains reach the end of their service life, the frequency of leaks and uncontrolled line failures become more common. These failures may result in unacceptable environmental incidents and lack of adequate potable water and sewage service that create unsanitary conditions for system users. Hot tapping and plugging may be an effective enabling procedure that allows the replacement, repair or rerouting of concrete pressure water lines and sewer force mains while maintaining operations. When necessary for continued operations a temporary bypass line is installed to permit continuous flow during the repair or modification (Figure 1).

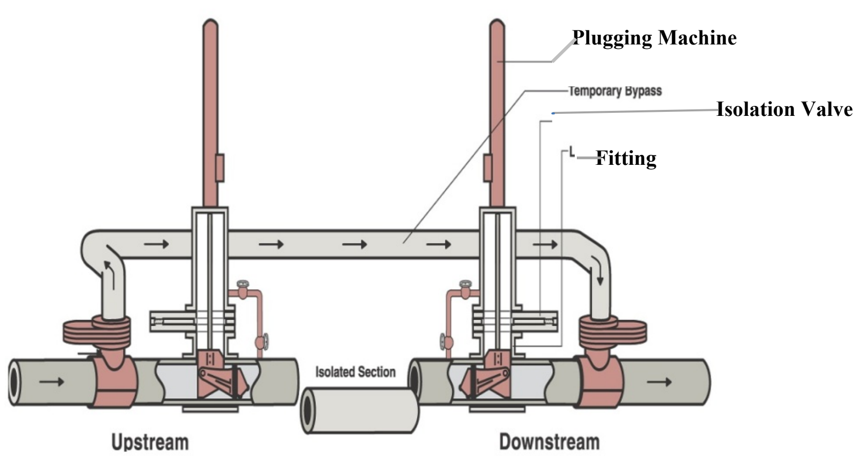


Figure 1 Plugging heads and temporary bypass conducting flow around isolated pipe section

TAPPING AND PLUGGING THROUGH A FITTING TO DIVERT FLOW

In order to insert a plugging head into a pipeline to isolate a section it is necessary to tap into the line, insert the head to block flow and, once work is complete, withdraw the plugging head and seal the opening created in the pipeline. The sequence for tapping and plugging through a fitting consists of:

1. Installing a plugging fitting around the pipe to be tapped (Figure 2) and mounting a temporary isolation valve and tapping machine above the fitting (Figure 3).

Figure 2 Plugging fitting installed

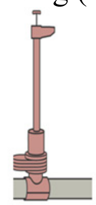


Figure 3 Isolation valve and tapping machine

2. Opening the isolation valve, lowering the cutter through the isolation valve, cutting and extracting a coupon (Figure 4), closing the isolation valve, removing the tapping machine and mounting the PLUGGING machine (Figure 5) on the isolation valve.

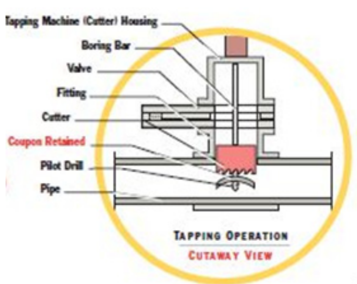


Figure 4 Tapping operation

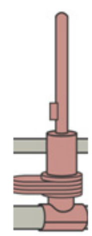


Figure 5 Plugging machine installed on valve

3. Opening the isolation valve again, inserting the plugging head to isolate a section of pipe (Figure 6), withdrawing the plugging head when work is completed on the section of pipe, closing the isolation valve and removing the plugging machine.

4. Placing the tapping machine on the isolation valve, opening the isolation valve, inserting the completion plug, removing the tapping machine, removing the isolation valve and installing a blind flange (Figure 7).

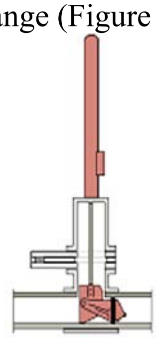


Figure 6 Plugging head in pipe

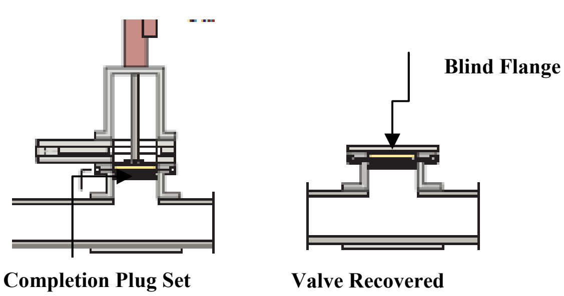
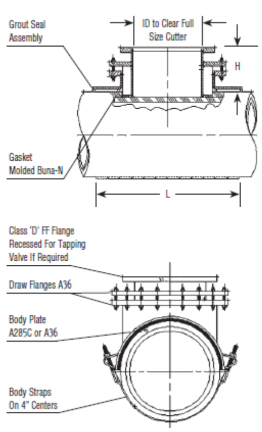


Figure 7 Completion plug set and blind flange installed

CONCRETE PRESSURE PIPE CHARACTERISTICS

Designing the fittings and plugging head for concrete pressure pipe requires determining certain pipe characteristics as well as pipeline system operating and control parameters. As part of the design of the saddle fittings (See Figure 8) it is necessary to determine the type and geometry of the pipe. The American Water Works Association (AWWA) defines three types of Concrete Cylinder Pipe. AWWA 301 defines prestressed concrete embedded cylinder pipe (Figure 9) and prestressed concrete lined cylinder pipe (Figure 10). AWWA 303 defines bar wrapped concrete cylinder pipe (Figure 11). Additionally, AWWA 302 defines reinforced non-cylinder pipe which only has reinforcing (Figure 19).

Concrete embedded cylinder pipe consists of: a welded thin gauge steel cylinder, with a steel bell joint ring welded on one end and a spigot ring joint welded on the other end; a concrete core; high tensile strength pre stressing wire and a dense cement mortar coating over the wire. The pipe is manufactured using vertical molds within which the cylinder with joint rings is placed. Concrete is then poured and consolidated within the mold. The wire is then wrapped and stressed over the core and the mortar coating is applied over the wire to complete the final pipe cross section



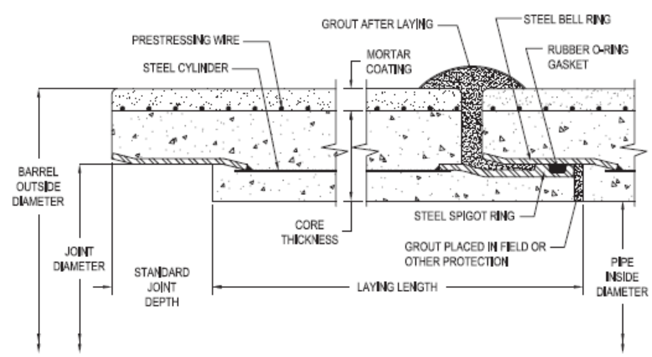
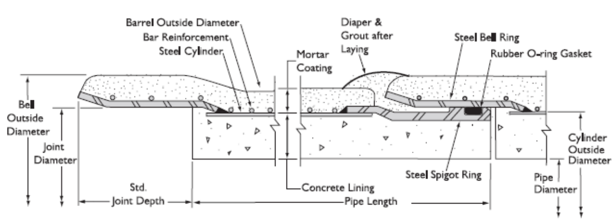


Figure 8 Concrete cylinder pipe tapping fitting Figure 9 Section concrete embedded cylinder pipe



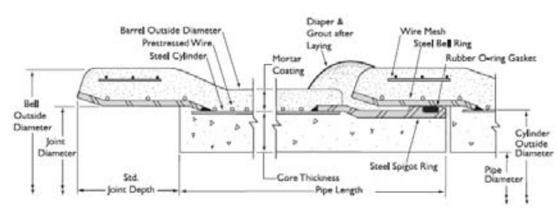


Figure 10 Section concrete lined cylinder pipe **Figure 11 Section concrete bar wrapped cylinder pipe**

Concrete lined cylinder pipe essentially contains the same elements as the embedded cylinder peep. However, the fabrication process is distinct. Lined pipe is produced by centrifugally casting concrete in the interior of the steel cylinder to form a core. The core is then wrapped with a prestressed wire directly on the steel cylinder and is then covered by cement rich mortar placed under high velocity to produce a dense outer coating.

Bar wrapped cylinder pipe resembles lined cylinder pipe except that a reinforcing bar is wrapped against the cylinder instead of prestressed wire before the outer mortar coating is applied. Production sequence for the bar wrapped pipe is the same as for the lined cylinder pipe.

PIPE CHARACTERISTICS AND SYSTEM PARAMETERS FOR FITTING DESIGN

Determining the outside diameter and roundness of the pipe is necessary to define the curvature of the saddle for each fitting to be used on the project. In addition, it is necessary to define the cross section of the pipe and curvature of the thin embedded steel cylinder, which may move out of circular shape during pipe production. The actual curvature of the cylinder is required so that the fitting inner gland pressure plate can be shaped to achieve a complete seal when drawn down against the steel cylinder. In order to determine the steel cylinder curvature the outer mortar coating has to be removed to expose the cylinder to scribe a line reflecting the curvature onto a template.

In certain instances it may not be possible to obtain a template of the cylinder curvature because a particular pipeline owner may be reluctant to chip off the mortar and expose the pipe due to concerns that the pipe might be damaged to the extent that service could be jeopardized. In other instances, the prestressed wire may be so closely wound about the cylinder that it would be necessary to cut the wires in order to make the template, thereby compromising or reducing the available operating pressure the pipeline can sustain for some period of time until the tapping saddle can be completely installed.

As an alternative to measuring the curvature, an outside cylinder diameter and curvature for a typical pipe can be assumed for purposes of fitting fabrication. As a contingency, in case the curvature of the cylinder differs too greatly from the assumed curvature, flat pressure plates, that can be molded and attached to the inner fitting gland close to the tapping site, can be shipped along with the fittings. Sending the pressure plates along with the fittings rather than make the plates only if necessary, can eliminate the possibility of waiting some time to produce and ship them at a later date.

IMPORTANCE CYLINDER GAUGE, PIPE DIMENSIONS AND CROSSECTION

The determination of the steel cylinder gauge and pipe wall cross section in conjunction with operating system parameters and controls is required to establish the allowable operating pressure during the fitting installation because once the pre stressing wires or reinforcing bars are cut and until the gland is completely installed the cylinder will act alone to resist internal pipe pressure. Exceeding this reduced allowable operating pressure may be a cause for pipe failure.

The reduced allowable pipe pressure also defines the maximum allowable testing pressure to confirm that the fitting gland is sealed properly against the cylinder itself. If the external pressure imposed upon the cylinder during gland installation and pressure testing exceeds the reduced allowable internal pressure, the cylinder may deform, thereby fracturing the inner core of the pipe, which may then interfere with the plugging operation or produce problems when cutting into the cylinder during tapping operations. Therefore, it is important to understand and know how to control or work around the particular system operating parameters to avoid the risk of exceeding the reduced allowable operating pressures during fitting installation.

In addition to understanding the pipe cross section for fitting installation, a successful pipe plugging operation requires actual inside diameter of the pipe in order to achieve a good seal around the inside circumference of the pipe. During the plugging operation the head that is in the machine housing in a vertical position moves down into the pipe and pivots from a vertical position to a horizontal position. In this position the sealing element installed on the perimeter of the head, manufactured in accordance with the calculated or measured inside diameter of the concrete core, closes the slight gap between the steel disk of the plugging head and the inside surface of the concrete pipe core as pressure increases behind the plugging head.

RESISTING FORCES DEVELOPED BY INSERTION OF PLUGGING HEAD

Overturning and thrust forces will be developed by the insertion of the plugging heads into a transmission line due to the geometrics particular to the equipment being used (Figure 12). The characteristics of concrete pressure pipe joints only allow for minimal movement during plugging operations. Therefore, much care must be taken in devising an adequate thrust restraint system. Depending on the geotechnical characteristics of the soil where the plugging operation is to take place large thrust blocks may be needed. For example, Figure 13 shows the forces developed by equipment plugging a 78 inch concrete embedded cylinder pipe operating at a maximum of 50 psi forces located along a canal in sedimentary soil strata and a cross section of the thrust restraint system used to limit movement.

Thrust Forces

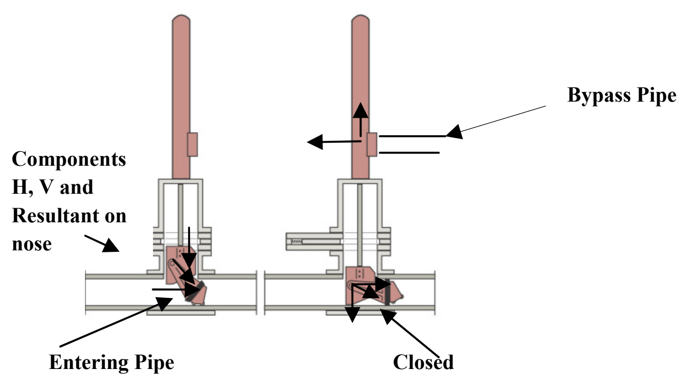
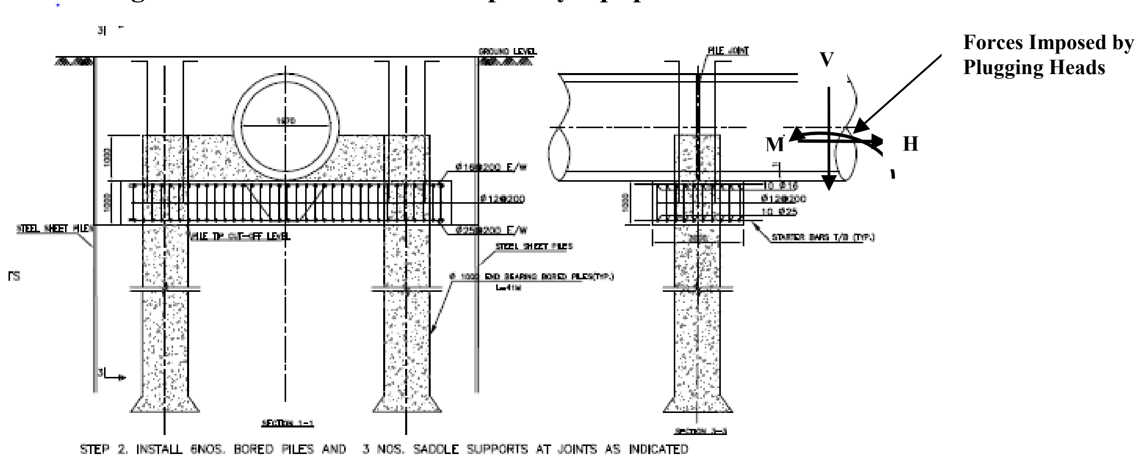


Figure 12 Thrust forces developed by equipment



Point	Dia. In.	H Kips	V Kips	M Kip Ft
F	78	243.57	56.52	852.51
D	60	144.13	39.41	393.35
C	48	92.24	39.41	199.85

Figure 13 Section thrust block and forces imposed by plugging heads at each location

CASE STUDY 1 REPLACEMENT OF A 66 INCH SEWER FORCE MAIN

Introduction

A 66-inch diameter prestressed concrete embedded cylinder pipe wastewater force main in a Midwest city failed four times since installation creating costly environmental incidents of raw sewage discharge into nearby bodies of water to the detriment of the environment and nearby industries. A condition assessment indicated that two miles of the pipe had an unacceptably high risk of future failures and required replacement. Since there was no alternative route to transport the sewage, the replacement had to be accomplished without interruption of service. A tapping and plugging procedure was selected to meet this objective.

Route Selection and Design

Three routes for the replacement of the pipe section with a high risk of failure were studied (Figure 14). Each of the routes started at Pump Station C.

Route 1 was under paved streets, and involved four horizontal directional changes. This route crossed the existing pipeline in one location. Although the construction costs were deemed higher than the other two routes selected for study, this route presented the least long-term risk for additional failures and, for this reason, upon analysis of the other two options, it was selected for the installation of the replacement pipe.

Route 2 was also under paved streets for the entire route with a minimum of bends, but crossed the existing pipeline and passed through an estuary area with poor soils that contributed to the previous failures of the existing pipeline. This route was rejected because of the long-term risks of environmental incidents that might negatively affect the estuary area.

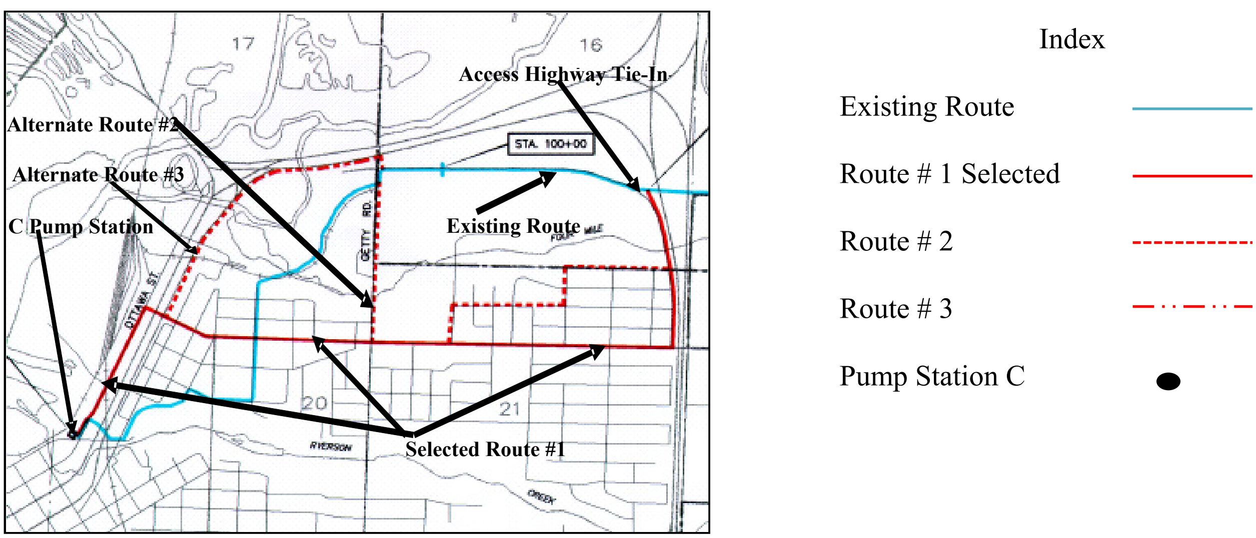


Figure 14 Alternative pipeline route study

Route 3 did not involve crossing the existing pipeline and was located under unpaved state right-of-way. However, the soils were very wet and soft and were the same types that contributed to the failures of the exisitng pipeline. This route was rejected due to the construction costs associated with the soft wet soils and the aggressive nature of the same.

Pipeline Design

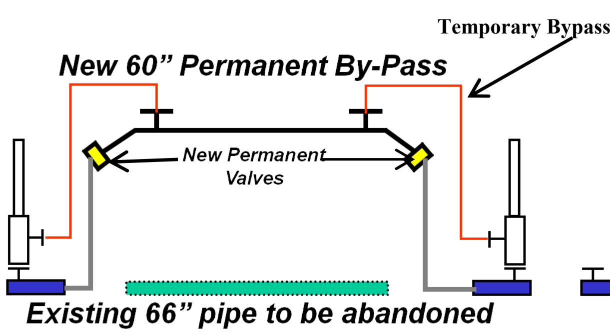
Alternative designs for the replacement of the pipeline were performed in accordance with American Water Works Association procedures. Considerations included available pipe sizes, energy costs for operation, surge analysis, thrust restraint and corrosion protection. The final options for contractor proposals were either prestressed concrete embedded cylinder pipe or ductile iron pipe. The new pipeline design goals were as follows:

The existing pipeline did not have restrained joints at the proposed connection point of the replacement pipe because there were directional changes with thrust restraints in the vicinity. However, stopping the flow for connection of the replacement piping, while keeping the pipeline in service, would create a large longitudinal force, which would act against the plugging head and tend to disjoint the existing pipeline. To counteract this force of over 135,000 pounds at the pipe's normal working pressure of 40 psi and 200,000 pounds at the design pressure of 60 psi, a large concrete thrust block 7 feet deep and 17 feet by 24 feet in area was designed to be installed around the pipeline once the tapping fittings were installed.

Tapping and Plugging Procedure

In preparation for putting the replacement pipeline in service, the system operating parameters were defined and plans to monitor and control these parameters within acceptable limits during the tapping and plugging procedure were developed. The schematic in Figure 15 illustrates the scheme for the procedure that makes use of the permanent replacement pipe to bypass sewage around the existing pipe while the interconnection of the replacement pipe is completed. The new permanent valves are closed prior to setting the plugging heads in the line to divert flow into the new pipe by means of the temporary bypasses. The schematic in Figure 16 illustrates the configuration once the replacement pipe is connected through the new valves and the tapping and plugging equipment is removed from the pipeline.

As the first step in the tapping procedure the replacement pipe was installed close to the connection points on the existing line, that would remain in place. The tapping and bypass fittings were installed on the pipe and the concrete thrust blocks were poured (Figure 17). The isolation valves and tapping machines were installed and the taps were performed without incident. During the tapping procedure, the inner core of the concrete pipe was found to be thicker than anticipated, thus requiring installation of a smaller sealing element to achieve the required seal. Once the smaller sealing element was installed, the plugging operation proceeded without incident. The bypass between the new pipe and the remaining prestressed concrete embedded cylinder pipeline was put into operation (Figure 18) and the replacement pipe was connected without interrupting flows or any leakage.



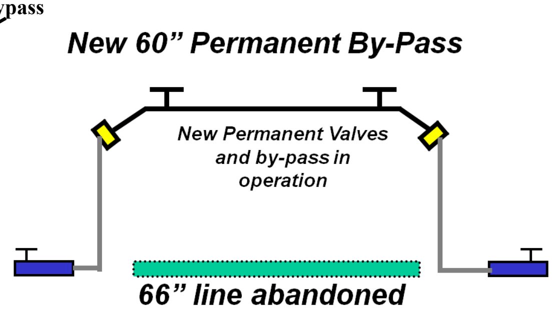


Figure 15 Scheme to bypass existing pipe Figure 16 Replacement pipe in service plugging complete

Figure 17 Tapping fittings and thrust block

Figure 18 Plugging machine and bypass in operation

The deteriorated pipe was then drained and taken out of service. The entire project was completed on schedule and within budget.

CASE STUDY 2 REPLACEMENT OF A CONCRETE WATER SUPPLY LINE

Background

Failures in a 54-inch diameter non-cylinder concrete pressure pipe supply line installed close to the Mississippi River were creating water quality problems for a Midwestern city water supply utility with an average daily demand of 10 million gallons per day. The system needed to have sections of the pipe replaced without loss of water to homes and businesses before the next spring flood season. In the fall of the year, a request was made to a specialized service company to investigate and submit a proposal to resolve the problem.

Process to Address and Resolve the Problem

Once the specialized service company was contracted to perform the work, two54-inch x 6-inch test taps were made to verify inside pipe geometry and integrity. After verification of the pipe geometry and integrity two 54-inchx 36-inch bolt on tapping fittings were manufactured for the project. With the support of a local construction company, the service company installed these fittings on the pipeline in order to isolate the section of pipeline to be replaced and rerouted. Since the pipe was the non-cylinder type (Figure 19), there was no surface against which a gland could be pulled down to create a seal. Therefore, a fitting without a gland was selected and it was decided to encase the fittings in concrete blocks (Figure 20) to minimize any fitting movement that could possibly result in any water leaking around the fitting and pipe surface interface.

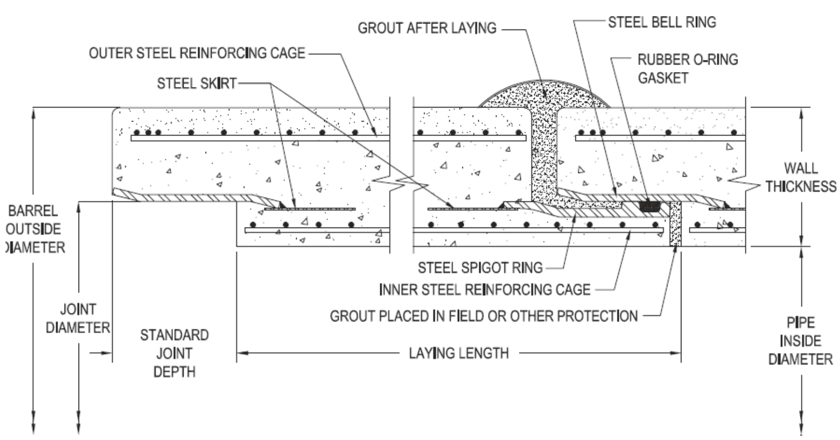


Figure 19 Non cylinder concrete pressure pipe

Taps were performed and the plugging machines were installed to divert the flow through a 36-inch diameter temporary bypass line connected to the tapping machine housing using a piping configuration similar to that described in Case Study 1. The new pipeline was connected and made operational (Figure 21) as the plugging heads were withdrawn.

Figure 20 Fittings and connection encased in concrete

Figure 21 Connecting new pipeline

During the course of the several days required to replace the defective sections of the 54-inch pipe the temporary 36-inch by-pass kept the City water system operational thereby maintaining normal water usage for homes and businesses as well as for schools, fire protection and hospitals.

When the new section of pipeline was made operational, the temporary bypass was depressurized, plugging machines and temporary isolation valves were removed and tapping fittings were sealed with completion plugs and blind flanges. If there would be a need in the future, the tapping fittings can be accessed for additional plugging operations.

CONCLUSIONS

Tapping and plugging procedures can be an effective technique to enable maintenance, repair or replacement of pipelines when they show signs of failure. Using the technique allows these projects to be accomplished on schedule and within budget without interruption of important services or the occurrence of environmental incidents.

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TDW Services, Inc. Project Archives, Tulsa, OK

Evaluating Chloramine Loss in Raw Water Supply Pipelines

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Abstract

Monochloramine loss was studied in two, approximately 70-mile pipelines within the Tarrant Regional Water District (TRWD) raw water supply and transmission main system. Both bench-scale studies and full-scale sampling were used to determine the impact of several factors that may affect monochloramine loss in the pipelines. The conditions of bench-scale study were representative of the range of water quality conditions encountered at the pump stations. Bench-scale results were compared to full-scale samples taken along the pipeline. Samples collected along the 70-mile pipeline were measured for chloramine concentration, pH, dissolved oxygen, as well as parameters known to indicate nitrification such as nitrite and free ammonia. Samples collected along the pipeline were also filtered with a 0.2 μm filter. Filtering the samples removed any nitrifying bacteria potentially present. Comparing the chloramine decay between the filtered and unfiltered samples allowed the affect of nitrification in the pipeline to be observed.

BACKGROUND

The Tarrant Regional Water District (TRWD) raw water supply and transmission system is rapidly growing to meet the demands of a growing population. Currently 1.8 million water users in eleven counties of North Central Texas are served by TRWD and customer cities. Water demands have tripled in the last 40 years and continue to rise. As demands and flow rates increase, biofilm growth in the pipelines may cause a reduction in pipeline capacity and increase in pumping costs due to increased friction losses. In response, biofilm control through the addition of chloramines at the lake pump stations was implemented, typically fed from March through November of each year when biofilm growth is at its peak. Chloramines were selected over other alternatives due to their forming a long-lasting residual disinfectant with minimal formation of disinfection by-products. However, since implementing chloramine feed at the lake pump stations, higher than anticipated monochloramine loss was observed in the pipelines. Monochloramine can decay through physical/chemical (autodecomposition) or biological processes, such as nitrification, which is a common challenge encountered by drinking water utilities that use monochloramine as a distribution system residual disinfectant.

Nitrification is a two-step biological process during which ammonia is converted to nitrite by ammonia-oxidizing bacteria (AOB), then to nitrate by nitrite-oxidizing bacteria (NOB). In addition to increases in nitrite and potentially nitrate concentrations, nitrification often leads to decreases in disinfectant residual, which can promote the growth of bacteria and leave the pipeline unprotected against potential contaminations, such as zebra mussels.

METHODS

Chloramine loss was studied in two pipelines within the TRWD system: (1) the Cedar Creek Reservoir and pipeline, and (2) the Richland Chambers Reservoir and pipeline. Each pipeline is over 70 miles long and is constructed of prestressed concrete cylinder pipe (PCCP). A schematic of the pipelines and sampling locations is provided in Figure 1.

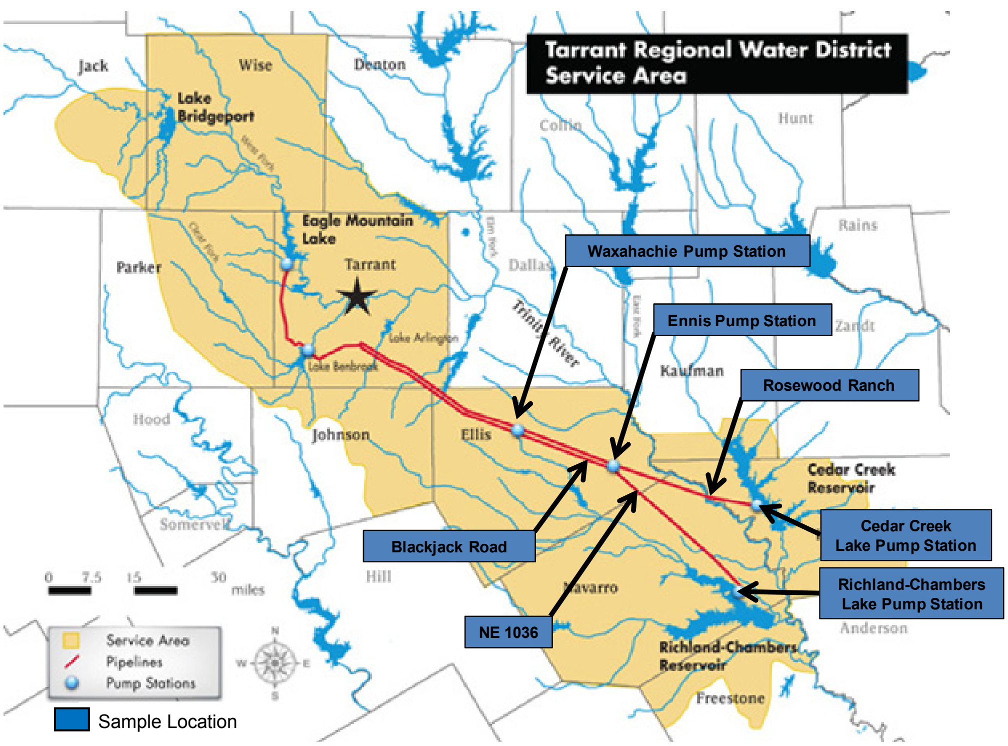

Figure 1. Pipeline and sampling locations.

Bench-scale chloramine formation and decay

Water was collected from Richland Chambers and Cedar Creek Reservoirs. Prior to testing, the pH of the water was adjusted with strong inorganic acid or base. Chloramines were formed by adding chlorine and ammonia simultaneously. Chlorine was dosed with a 3,000-5,000 mg/L stock free-chlorine solution prepared by diluting sodium hypochlorite with distilled (DI) water. The concentration of the stock solution was determined prior to use. A 1,000-2,000 mg/mL as N stock ammonia dosing solution was prepared by diluting liquid ammonium sulfate in DI water. Samples were incubated in amber glass bottles, capped headspace free with Teflon-lined septa. Total chlorine residuals were measured using Hach Method 8167, and monochloramine/free ammonia residuals were measured using Hach Method 10171 and 10200. The chloramine dose was selected to match current operating conditions to provide comparison to full-scale data. Once dosed, the samples were measured for total chlorine, monochloramine, and free ammonia at varying times that corresponded to the anticipated detention time in each of the pipelines under anticipated flow conditions to allow for comparison to the full-scale data collected along the pipeline.

Full-scale chloramine decay evaluation

Chloramine formation and decay was evaluated in the full-scale pipelines. Approximately 1 liter samples were collected at various locations in the raw water pipeline (Figure 1). Approximately 500 ml of sample were filtered with a 0.2 μm filter. Filtering the sample removed the nitrifying bacteria potentially present (Sathasivan et al., 2006). Both raw water and filtered water samples were analyzed for monochloramine, total chlorine, free ammonia, nitrite, and nitrate.

RESULTS AND DISCUSSION

Key water quality parameters for each source water are presented in Table 1. Both bench-scale studies and full-scale sampling were used to determine the impact of the many factors that may affect monochloramine loss, including water quality (pH, total organic carbon), water age, the process of chloramine formation (i.e., chlorine to nitrogen ratio) at the lake pump stations (LPS), and nitrification. Bench-scale results were compared to full-scale samples taken along the Cedar Creek pipeline (Figure 2) and the Richland Chambers pipeline (Figure 3). In addition, a sample (identified in Figure 2 and Figure 3 as the hold sample) was taken at the effluent of each lake pump station and held for a detention time similar to those anticipated in each pipeline. The bench-scale experiments were dosed with chlorine and ammonia at the same concentration as full-scale. However, significant differences in monochloramine concentrations were observed immediately after chemical addition. Higher concentrations of monochloramine were formed in the bench-scale experiments than observed full-scale. Once formed, the rate of monochloramine decay was similar in the bench-scale experiments and the hold sample. The observed difference in initial monochloramine concentration may be a result of differences in chemical mixing conditions between bench- and full-scale.

Table 1. Key water quality parameters.

	Cedar Creek			Richland Chambers		
Parameter	Raw	LPS Effluent	BS	Raw	LPS Effluent	BS
pH	8.0	8.8	8.3	7.8	7.9	8.3
TOC (mg/L)	6.1	--	--	4.6	--	--
Alkalinity (mg/L as $CaCO_3$)	68	--	--	94	--	--
Temperature (°C)	31	--	--	30	--	--
Turbidity (NTU)	9	--	--	6	--	--

Notes: LPS = lake pump station; BS = bench-scale; -- indicates not measured

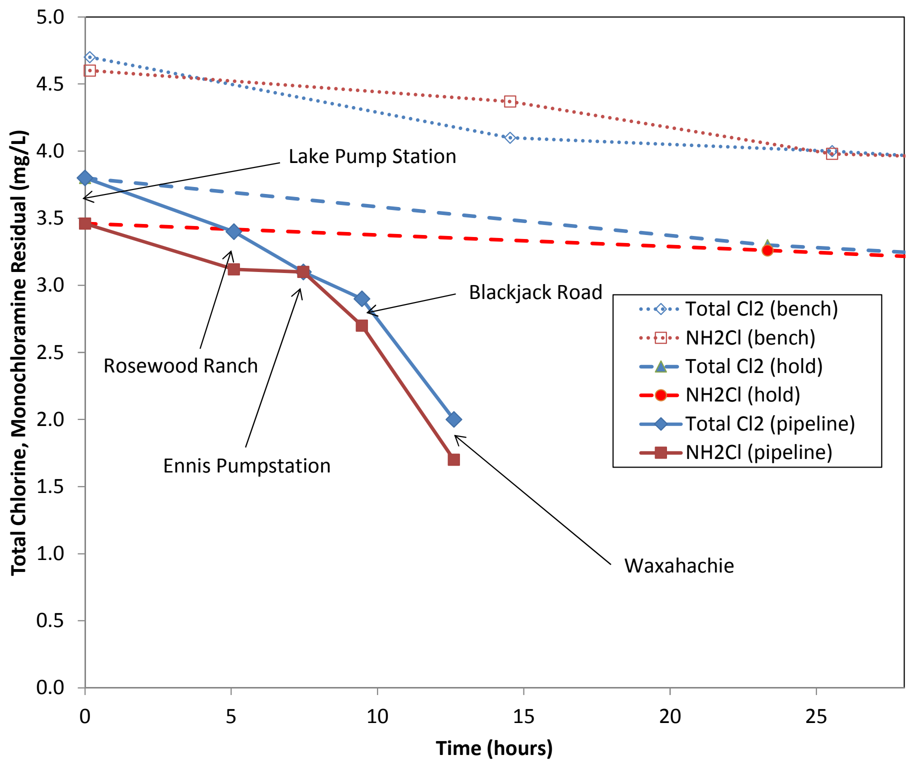

Figure 2. Chloramine decay: Cedar Creek.

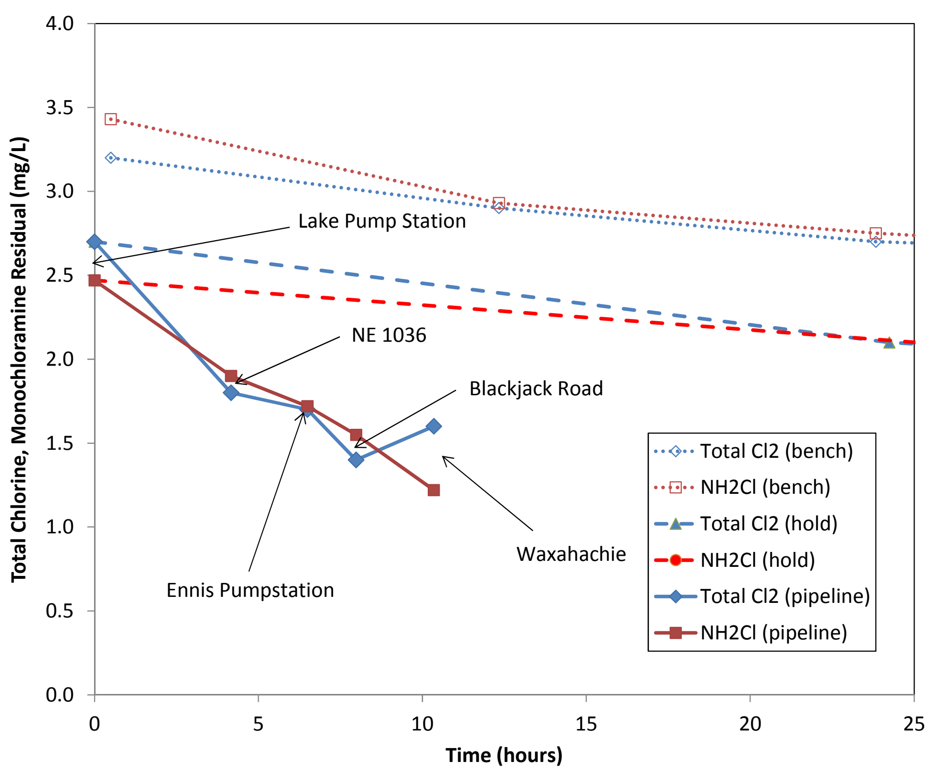

Figure 3. Chloramine decay: Richland Chambers.

Significantly greater monochloramine decay was also observed in the full-scale pipeline than in the bench-scale studies or hold sample (Figure 2 and Figure 3). This loss may be attributed to nitrification. To determine if the increased loss observed in the pipeline was a result of nitrification, samples collected along the pipelines were measured for monochloramine, pH, dissolved oxygen, as well as parameters known to indicate nitrification such as nitrite and ammonia. These data are presented for the Cedar Creek and Richland Chambers pipelines in Figure 4 and Figure 5, respectively. Nitrite concentrations increased the further the samples were collected from the lake, whereas monochloramine and free ammonia concentrations decreased. These trends are indicative of nitrification in which ammonia is converted to nitrite by ammonia-oxidizing bacteria.

Samples collected along the pipeline were also filtered with a 0.2 µm filter. Filtering the samples removed any nitrifying bacteria potentially present (Sathasivan et al., 2006). Comparing the chloramine decay between the filtered and unfiltered samples allowed the presence of nitrifying bacteria in the pipeline to be verified. Data are presented for the Cedar Creek and Richland Chambers pipelines in Figure 6 and Figure 7, respectively. Less monochloramine decay was observed in the filtered samples than the non-filtered samples. Free ammonia concentrations also increased in the filtered samples as the monochloramine decayed, which is expected for chloramine decay in the absence of nitrification. In the non-filtered samples, free ammonia concentrations decreased and nitrite concentrations increased. The increased loss of monochloramine in the non-filtered samples when compared the filtered samples, loss of free ammonia, and increased nitrite concentrations are indicative of nitrification.

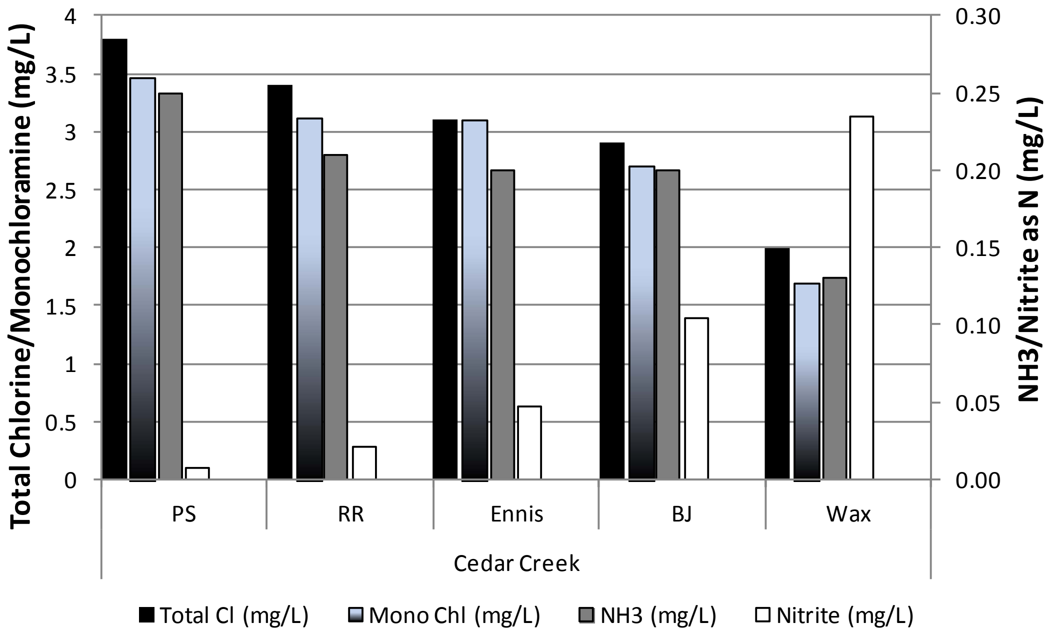

Figure 4. Nitrification indicators: Cedar Creek.

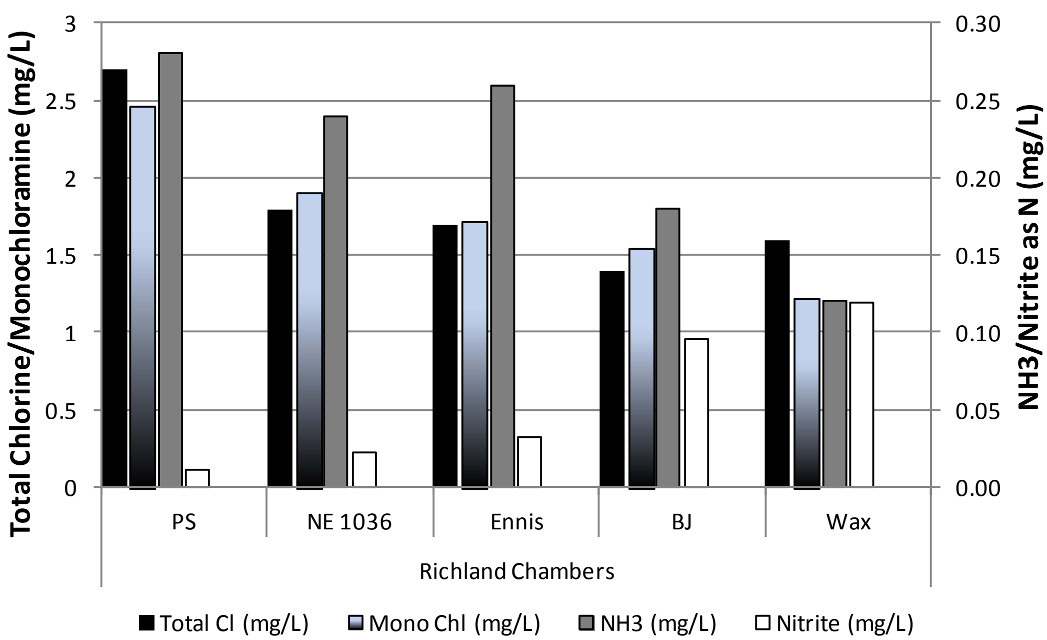

Figure 5. Nitrification indicators: Cedar Creek.

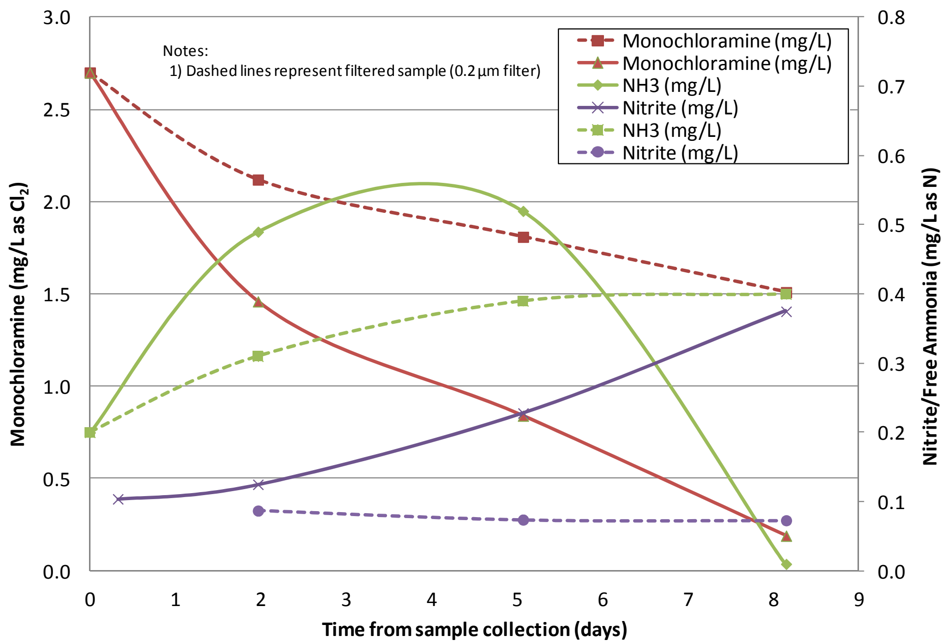

Figure 6. Nitrification evaluation in Cedar Creek Pipeline (Black Jack Road).

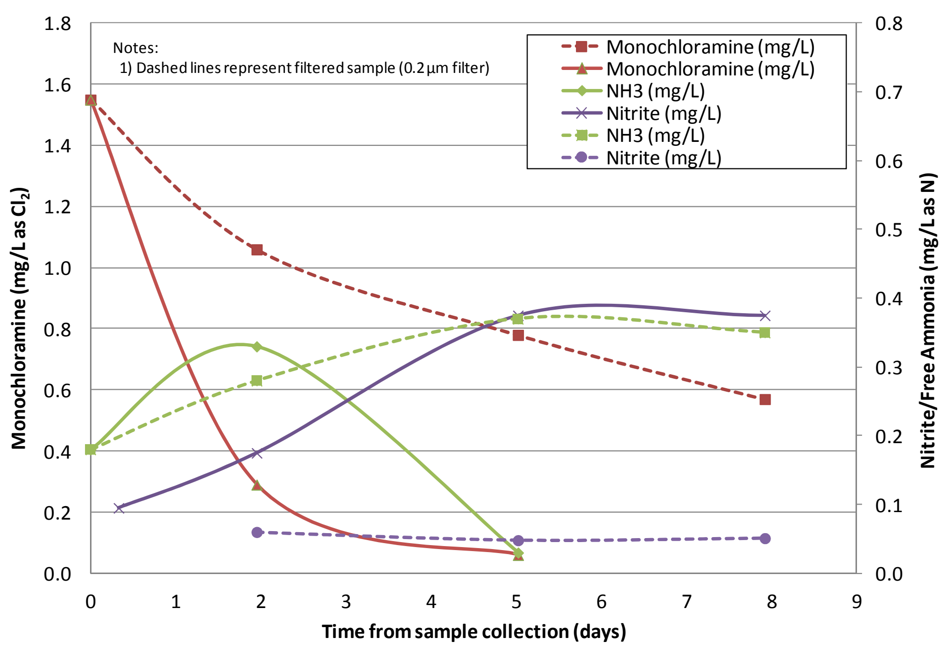

Figure 7. Nitrification evaluation in Richland Chambers Pipeline (Black Jack Road).

CONCLUSIONS

The TRWD operates two approximately 70-mile pipelines as part of its raw water supply and transmission main system. Biofilm growth in the pipelines may cause a reduction in pipeline capacity and increase in pumping costs due to increased friction losses. In response, biofilm control through the addition of chloramines at the lake pump stations was implemented, typically fed from March through November of each year when biofilm growth is at its peak. Since implementing chloramine feed at the lake pump stations, higher than anticipated monochloramine loss was observed in the pipelines. Bench- and full-scale studies indicated that a majority of the observed chloramine loss was a result of nitrification. Future studies will focus on strategies to limit nitrification within the pipelines.

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Evaluating the Effectiveness of the Sewer Root Control Program for the City of Baltimore

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Abstract

The Baltimore City Sewer Root Control Program was launched in 2007 to reduce root-related sewer overflows and basement flooding by using maintenance and chemical treatment activities. In mid-2014 the City had the third treatment cycle already scheduled, which prompted the execution of an effectiveness study to evaluate root regrowth, reduce the inventory of pipes in the Program, and establish future planning parameters. Due to the large number of historical inspection data in the study, a simplified analysis was adopted. The approach established a combined root condition classification using the National Association of Sewer Service Companies (NASSCO) Pipeline Assessment and Certification Program (PACP) terminology that included both the oldest and most recent inspections. The results of the study were useful for calculating a root control effectiveness value, as well as other significant indicators such as the percentage of pipes with no history of root problems and the percentage of reoccurring root balls.

ROOT CONTROL PROGRAM OVERVIEW

In 2002, Baltimore City (City) entered into a Consent Decree with the Environmental Protection Agency (EPA), Department of Justice (DOJ), and the Maryland Department of the Environment (MDE) for the wastewater collection system, which required the City to implement a Root Control Program that provides both short-term mitigation and long-term planning to address root infestation. The primary goal of the Root Control Program is to reduce the occurrence of Sanitary Sewer Overflows (SSOs) and basement flooding in the collection system caused by root-related blockages, e.g. mainline chokes and house connection chokes. The Root Control Program seeks to achieve this goal through the following: scheduling root control treatment in a prioritized, systematic fashion; developing a clear strategy regarding the proactive maintenance, rehabilitation, and replacement of pipes impacted by root intrusion; coordinating the application of root control in laterals with the City's Lateral Assessment and Renewal Program; and integrating Asset Management System tools to track and address root regrowth and drive quality assurance in the long-term.

The initial inventory of pipes in the root control program was developed in 2007 based on review of the CCTV inspections performed under the City's sewershed studies. Beginning in 2012, the precise locations of root observations from the Closed Circuit Television (CCTV) surveys were plotted and then overlaid in GIS with work order and service request data from Cityworks to select specific streets and neighborhoods to target for root control. The scope of the program grew as analysts investigating SSOs and basement flooding would review newer CCTV data collected by Department of Public Works (DPW) Maintenance Division in response to service requests. The analysts would determine which pipes were suspected to have caused loss of service and flag them for inclusion in the Program if they contained roots and had not been previously treated. Future treatment of pipes will be prioritized to target pipes that have either caused chokes but have never been treated, and pipes that are receiving a second follow-up treatment. Pipes that have already been treated twice will be placed into a three-year cycle where all pipes within each sewershed sub-basin will be treated at the same time once every three years.

Beginning in FY 2014, the City began an aggressive campaign to begin treatment of laterals in addition to mainline sewers. Laterals were selected based on the criteria of a history of previous chokes causing basement flooding as well as CCTV showing roots. The current root control inventory contains approximately 2,500 laterals. Future lateral treatments will be directed by the Lateral Inspection and Renewal Program, which is expected to eventually be treating over 3,500 laterals annually by 2021 to prevent root growth.

Over time, as mainline and lateral sewers are renewed or replaced, the inventory will gradually shrink; however, some pipes may remain within the program indefinitely as the most cost-effective means of maintenance. Success of the program will be determined by a reduction in the number of root related SSOs, as well as a decrease in the number of choke events occurring on laterals and mainlines treated within the program.

STUDY PURPOSE AND SCOPE

The objective of the study was to evaluate the effectiveness of the Root Control Program on sewer mains using historical CCTV to propose a more effective treatment cycle.

A sample set of approximately 460 sewer segments, with diameters ranging from 8 to 12 inches and treated under the Root Control Program, were reviewed in this study. These mainline segments have all received two or three treatments of root chemical and were soon due for the next treatment per the current contractor warranty-based treatment cycle. For each segment, the study required reviewing 'old' CCTV videos (e.g. from 2007 or 2008) against the 'recent' CCTV videos (e.g. from 2013 or 2014). The study sample included approximately 1,300 CCTV videos.

APPROACH

To analyze such a large number of segments, most of which have two to four CCTV video records and variable root growth through the length for the segment, a simplified analysis was adopted. The approach consisted of analyzing oldest and newest videos for each pipe segment and representing pipe conditions before root treatment and after root treatment, respectively. The poorest root condition observed for each video was documented, regardless of where the root condition was found or how many of these occurred in the segment.

A simple classification for root mass intrusion was established and included three categories from best condition to poorest condition. To define each one of these categories, definitions from the National Association of Sewer Service Companies (NASSCO) Pipeline Assessment and Certification Program (PACP) for the Root group of codes were used. The conditions established for this study are shown in Table 1.

Table 1. Root mass intrusion condition classification

Root Condition Classification	Definition
None (N or n)	Denoting no roots.
Fine/Medium (FM or fm)	Any presence of roots from apparent no reduction to a 50% reduction of pipe cross-sectional area.
Root Ball (B or b)	A root mass creating a pipe cross-sectional area reduction greater than 50%.

For each pipe segment, the poorest root condition classification observed in each video (oldest and most recent) was recorded regardless of the location and the number of occurrences. Based on the oldest and most recent root condition classifications, a single combined root condition was developed to indicate changes over time. To differentiate before and after conditions, a lowercase letter for the oldest condition and an uppercase letter for the most recent condition were employed. For example, a combined root condition fmB describes a segment transitioning from *roots fine/medium* to *root ball*.

The condition fmFM includes pipes transitioning from Roots Medium to Roots Fine, as well as from Roots Fine to Roots Medium. This classification was needed to provide an manageable and less subjective condition cataloging. The single fmFM classification was also based on the premise that in most cases, Roots Fine and Roots Medium are addressed by maintenance units using a combination of root chemical treatment and jet cleaning.

A database was used to store the combined root condition for each segment. A special Microsoft Access form, shown in Figure 1, was used to input the data. Using the database records, a matrix was generated to summarize the full results of the study. The template for the root condition matrix is shown in Figure 2. The values for each combined condition were displayed in percentages.

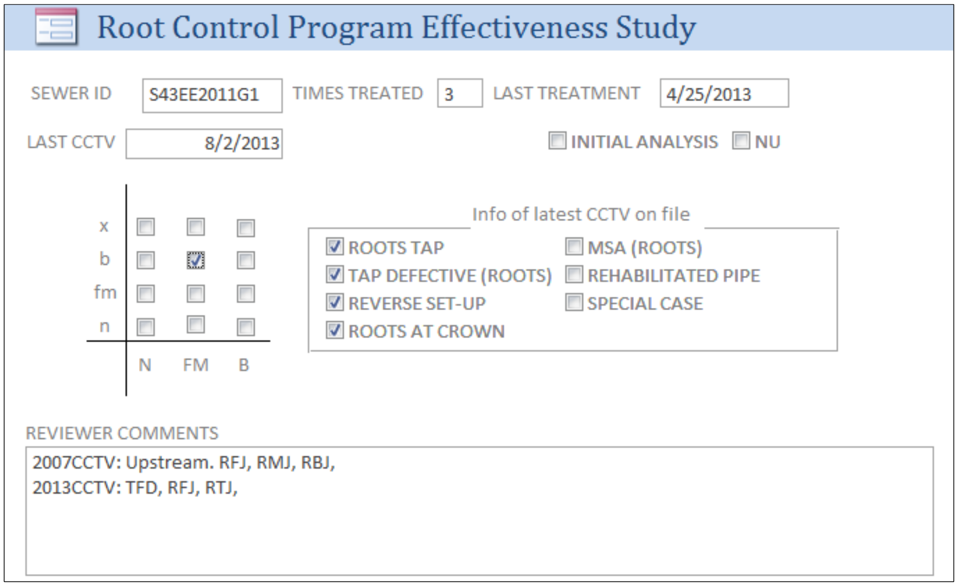

Figure 1. Microsoft access database form

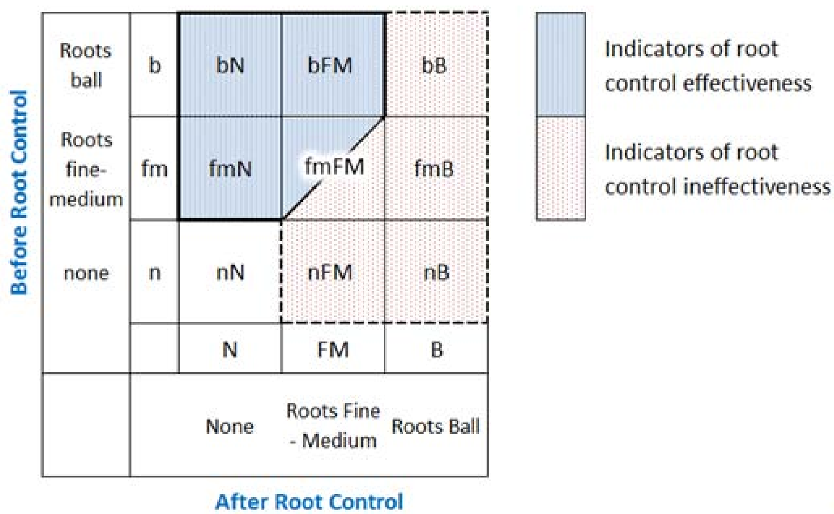

Figure 2. Combined root condition matrix

Using this matrix, an analysis for effectiveness of root control was developed and some assumptions formulated. Segments with improved root conditions (e.g., from root ball to fine/medium roots) were classified as indicators of root control effectiveness, whereas segments with non-improving root conditions (e.g. from none to fine/medium roots) were classified as indicators of root control ineffectiveness. The conditions that define root control effectiveness and ineffectiveness are shown in Table 2.

Table 2. Defining root control effectiveness

Indicators of Root Control Effectiveness	Indicators of Root Control Ineffectiveness
bN	nFM
bFM	nB
fmN	fmB
Half of the segments in fmFM	bB
	Half of the segments in fmFM

As shown in Table 2, for the condition fmFM, it was assumed that half of the segments transitioned from roots medium to roots fine, and half of the segments transitioned from roots medium to roots fine.

CHALLENGES

As the review of videos progressed, some anticipated and unanticipated challenges became clear. As explained in Table 3, some of the challenges reduced the sample size. As a result, the study computed results using 322 out of the 460 sewer segments.

Table 3. Observed Challenges

Challenges reducing the sample size	Challenges *not* reducing the sample size
• Video quality of CCTV videos. Fine roots are difficult to see when image is pixelated. • High flows that put a grease film on the lens. Usually when comparing inspections done at midnight with ones done between 7-9am. • Corrupt videos or server problems. • Inspections carried-out using a push camera. • Pipes rehabilitated after initial CCTV inspection.	• Reverse set-ups: comparing, for example, old videos going downstream against most recent videos going upstream. • Operator not panning or stopping at lateral connection to the sewer main. • CCTV recording containing two or more pipe segments. • Blockage problem is a combination of roots and grease, which most of the time covers all roots and prevents CCTV analyst from identifying the roots. • Old videos of lines not pre-cleaned may show grease covering roots. New videos pre-cleaned show those roots, therefore giving the impression that roots have started to grow.

RESULTS

All the CCTV videos (approximately 1,300) for the 460 pipe segments were reviewed. As discussed, the final number of pipe segments analyzed in the final matrix was 322 due to various challenges that reduced the sample size. Using the approach described, the resulting matrix was generated (see Figure 3).

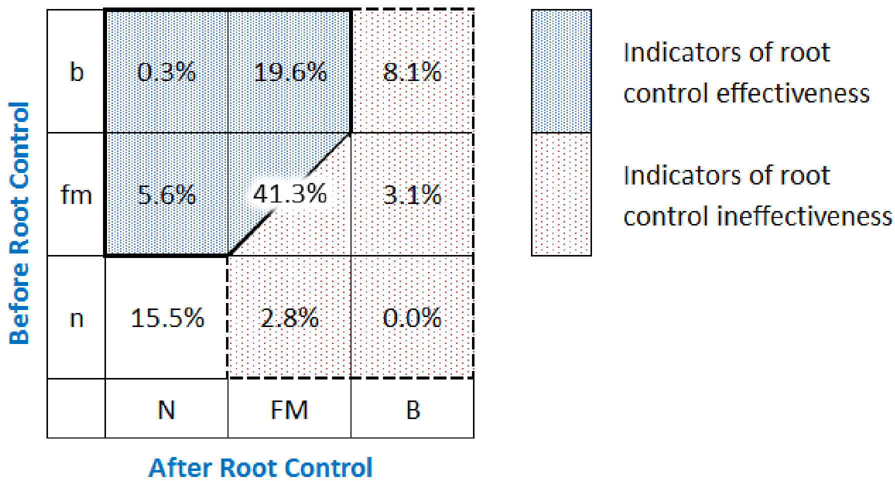

Figure 3. Root control effectiveness study resulting matrix

Based on these results, effectiveness calculations and other values were obtained. A summary of these values are shown in Table 4.

Table 4. Summary of Calculations

Value	Percentage
Effectiveness value (bN%+bFM%+fmN%+0.5fmFM%)	46% approx.
Ineffectiveness value (nFM%+nB%+fmB%+bB%+0.5fmFM%)	38% approx.
Pipes with no history of roots (nN)	16% approx.
Rehabilitated pipes	6% approx.
Pipes with root problems at tap connection	44% approx.

Before drawing any conclusions, the reader should note that this analysis considers *all* maintenance work related to reducing root growth or improving pipe structure. Therefore, the study does not distinguish between root chemical and other work that may have occurred in these mains, such as cleaning prior to CCTV.

One important positive indicator of root control effectiveness is the difference between percentages in the bFM condition against the fmB condition. There are approximately 16.5% (19.6%-3.1%) more pipe segments transitioning from Root Ball to *Roots Fine/Medium* (bFM) than from *Roots Fine/Medium* to *Root Ball* (fmB). The same holds true for the difference between the percentages in the fmN and nFM conditions. There are approximately 3% (5.6%-2.8%) more pipes transitioning from *Roots Fine/Medium* to *None* (fmN) than from *None* to *Roots Fine/Medium* (nFM). Table 4 also shows that approximately 16% of the pipes in the sample did not have history of root problems, yet they were part of the Root Control Program. Figure 3 shows that approximately 8% of the segments are still being affected by root balls (bB). Note that this analysis excludes rehabilitated pipes (representing approximately 6% of the segments), as these are no longer part of the Root Control Program; therefore, the results for the bN condition is 0%.

CONCLUSION

Given the results of the study, the City is focusing its attention on the fmB and bB cases, which account for approximately 11% of the sample, since these segments are more capable of producing SSOs and basement flooding. Lines with previous history of basement flooding or sewer main chokes are given special attention. The City has removed, from the Root Control Program inventory, all pipes with no history of root problems (nN) found in the study. The study also recommends including alternative methods to abate recurring root balls, determining an effective method for removing segments from the inventory, and considering a program for replacing problematic tree species.

The results provided a quantitative root control effectiveness value; however, the study was unable to provide enough data to determine a more effective chemical treatment cycle, since exact times for treatment, cleaning and CCTV cycles were not considered in this study. Starting in 2015, the City will conduct a more comprehensive and controlled pilot study to identify and establish a precise long-term root treatment cycle. In addition to sewer mainlines, the pilot study will evaluate the effective treatment cycle for sewer laterals.

The pilot study plans to conduct a review of condition assessment scores for a number of mainlines and laterals during a four-year and a three year period, respectively. During that period, usual root treatments will occur and CCTV inspections will follow at three months, six months and every six months after that. Condition scores generated using the PACP and Lateral Assessment and Certification Program (LACP) will give City engineers the means to identify the best possible root control cycle duration.

ACKNOWLEDGMENTS

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Performing a Condition Assessment of a 24-inch Diameter Gas Line Supplying an Important Part of a Suburban Area of a Large Midwest City

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Abstract

This paper discusses the inspection tools, procedures and field operations performed in 2011 for the in line condition assessment of a 30 year old, 13 mile long, 24 inch diameter gas transmission line, operating between 250 and 300 psi, that had not been inspected after being placed in operation. Since the line supplies an important part of a suburban area of a large Midwest city with natural gas for domestic, commercial and industrial consumption, the assessment was important to determine whether the line conditions were adequate for continuing operational safety. Additionally the assessment would serve as a comparative baseline for future assessments while the associated cleaning would help improve line flow. Project challenges included limited work areas, public activities adjacent to the tool launching site; load restrictions; limited access to the transmission line easement and right of way, that traversed both residential and environmentally sensitive areas; and commercial and light industry activities close to the gas venting lines at the launching and receiving site.

GAS TRANSMISSION LINE REQUIRES INSPECTION CONTINUING OPERATIONS

A company supplying natural gas to an important area of a large Midwest city required a condition assessment of a 13 mile long, 24-inch diameter gas transmission line, operating between 250 and 300 psi. The assessment was of particular interest because the company needed to determine if there were any anomalies or damage that would require immediate repair to avoid suspending energy supply for domestic, commercial and industrial consumption in a densely populated urban area that presented many challenges to the performance of the assessment. The challenges included confined work areas, public activities adjacent to the tool launching site; load restrictions on roads and at the launching sites; limited access to the transmission line easement and right of way, that traversed both residential and environmentally sensitive areas; and commercial and light industry activities close to the gas venting lines.

The company developed a scope of work to perform the initial condition assessment that included line cleaning, verification of minimum internal diameters, mechanical cleaning and in line and electromagnetic flux field measurements along the length of the pipeline in order to measure and locate pipeline features and anomalies. The assessment could in turn be used as a baseline in conjunction with future assessments to project rates of progressive changes in the pipeline condition, a useful maintenance planning tool. In addition, the cleaning associated with the preparation of the pipeline for the introduction of measuring tools into the line for the condition assessment would help improve operating flow. The energy company scheduled the condition assessment of the transmission line for the spring of the year, with preliminary results and field confirmation of reported anomaly locations shortly thereafter.

IN LINE INSPECTION TOOLS

In line inspection tools consist of a variety of devices that measure directly or indirectly anomalies in the pipeline geometry or pipe wall. These devices are in the great majority propelled through pipelines either by the pressure of the product the pipeline transports or by some other product injected into the pipeline, such as nitrogen. There are two types of instruments, direct and indirect. Direct sensing instruments use sensors that are in contact with the surface of the interior of the pipe. As the sensors travel over the surface the deflection of the contacts caused by variations of pipe geometry are registered in electronic memory storage. Certain types of indirect instruments generate electromagnetic fields ahead of sensors, not in contact with the pipe, as they travel through the interior of a pipeline. The sensors detect variations in the electromagnetic fields or magnetic flux leakage (MFL) caused by distinct anomalies and characteristics of a pipe such as valves and weld seams. The variations in the magnetic fields caused by anomalies and characteristics along the pipeline are registered in electronic storage units contained in the instrument.

Figure 1 shows a variety of tools designed and built for in line inspection (ILI). As the figure shows, the tools may be coupled to form a train of tools in order to determine, all in one run, the locations and measurements of the particular anomalies that each type of instrument is designed to best detect. The train usually consists of an odometer that measures the distance traveled along the pipeline, the tools and a battery to power the instruments.

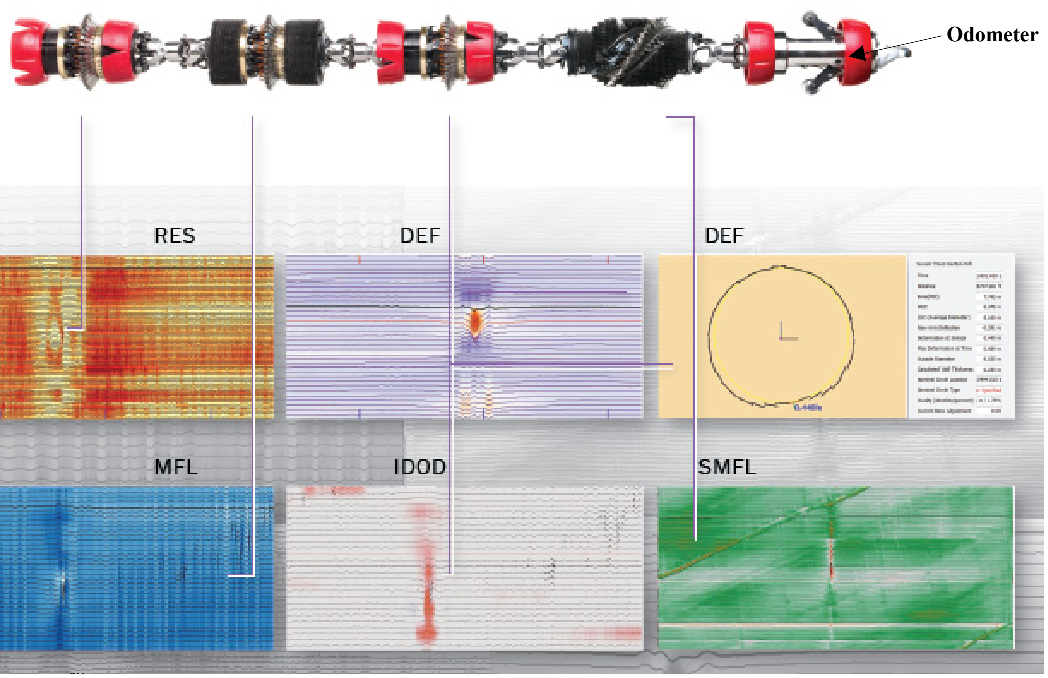


Figure 1 Different types of in line inspection tools

Geometric Magnetic Flux Leakage Tools

Geometric Magnetic Flux Leakage (GMFL) tools (Figure 2) of high resolution provide detection of internal and external metal loss and other metal anomalies. This type of instrument is specifically suitable for use in gas lines. It is used as a means to determine the degree and extent of corrosion existing on a line. If the corrosion of the pipe wall is found to be beyond acceptable thickness limits for the specified pipeline operating pressure, repairs will be required to maintain that operating pressure. If the corrosion proves to be isolated, then it may be feasible to perform a local repair. If the corrosion is extended over a longer length of the pipeline, then pipe removal and replacement will be necessary.

High Resolution Deformation Technology Tools

High Resolution Deformation Technology (DEF) tools (Figure 3) use sensors traveling on the pipeline surface to detect minimal changes in pipe surface geometry. The instrument uses high resolution technology to detect pipe dents and expansion, which can be analyzed to determine induced pipe strain. If the dents or expansions cause excessive strain beyond that acceptable for safe operations at the specified pressure, these deformations will require removal to restore the pipeline to safe operating pressure levels. Dents in the pipeline may also impede the passage of cleaning tools used during the normal course of pipeline operations and may require removal for this reason. DEF tools may be used simultaneously with other compatible technologies to obtain expanded data sets from a single tool run.

Figure 2 GMFL tool

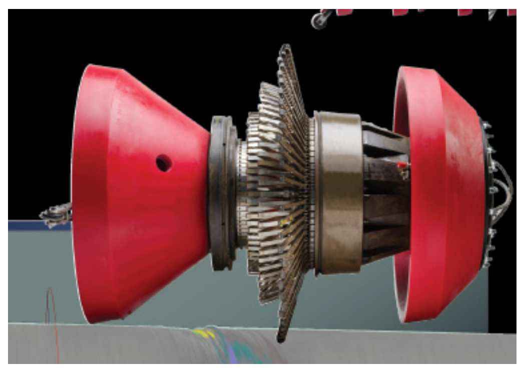


Figure 3 High resolution DEF technology tool

Metal Loss Inspection Tools

Solid magnet Magnetic Flux Leakage (MFL) tools (Figure 4) provide high resolution metal loss inspection that locates and sizes external and internal metal loss and other metallic anomalies. This type of tool has a minimum of mechanical parts that require maintenance and repair time. The information provided by these tools is analyzed to determine the location of repairs required to safely maintain specified operating pressure of the pipeline or the acceptable operating pressure of the pipeline if no repairs are made. A balance between the extent of repairs and the desired minimum safe operating pressure can be determined with the information provided by the tool.

Spiral Magnetic Flux Leakage Tools

Axial MFL tools are better at detecting anomalies and defects that are closer to being perpendicular to the axis while circumferential MFL tools are better at detecting anomalies oriented closer to being parallel to the axis of the pipeline. Figure 5 illustrates that this is because the axial electromagnetic flux lines, generated at 90 degrees to the circumference, are distorted in a greater proportion by anomalies closer to a circumferential orientation and the circumferential magnetic flux lines are distorted to a greater degree by anomalies oriented in an axial direction.

Spiral Magnetic Flux Leakage (SMFL) tools (Figure 6) combine the advantage of both axial and circumferential MFL detection abilities. It provides more accurate defect sizing and long seam sizing information, while not significantly adding to tool train length, which may allow passage of a tool train through smaller minimum pipeline curves.

XYZ MAPPING

XYZ mapping is three dimensional information developed using data from inertial guidance equipment that is included in the ILI tool train. The information derived from the inertial guidance system data is correlated with above ground GIS survey information to provide three dimensional location of the pipe centerline trajectory and of features such as valve, girth weld and anomaly locations. This is a very useful option when considering the need to locate anomalies requiring repair in order to maintain pipeline operating integrity.

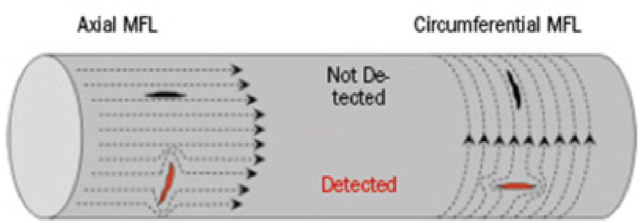


Figure 4 Solid (MFL) Figure 5 Anomalies detected by axial and circumferential flux lines

Figure 6 Spiral Magnetic Flux Leakage (SMFL)

PREPARING A PIPE FOR AN IN LINE INSPECTION TOOL RUN

Prior to performing an In Line Inspection tool run it must be ascertained that the pipe has no bends with radii smaller than that specified by the service provider for the combination of tools to be passed through the pipeline. If there are bend radii smaller than allowable, either the tool train must be shortened and the number of runs increased to collect the desired data, or the bends must be removed and replaced with bends of greater radii. Usually the choice between these two options depends upon the economics of the two options in conjunction with future maintenance and operational requirements associated with future cleaning to maintain minimum flow rates.

In addition to assuring the bend radii meet the minimum tool configuration requirements, it is necessary to determine if there are any major obstructions within the pipeline line that would not allow the passage of the ILI tool. As a first step, soft foam pigs (Figure 7) slightly smaller than the inside diameter of the pipe are propelled through the pipeline. If these pigs pass through the pipeline without damage, one can proceed to clean the interior surface of the pipe to eliminate any material that might impede the proper functioning of the inspection tools. Dense foam pigs with a hard outer surface with the ability to clean the interior of the pipeline are used for this purpose.

After foam pigs traverse the pipeline, a gauge pig (Figure 7) with a gauge consisting of a thin plate with many lobes, usually machined to 95% of nominal inside pipe diameter, is passed through the pipeline. If the gauge plate emerges without gouges or bent lobes, the pipe is deemed free of deformations that would block passage of the ILI inspection tools.

Following the passage of the gauge pig, a brush and magnetic pig, or combination of these two, may be run through the pipe to scrape off additional material from the pipe wall as well as to pick up any stray metallic objects, such as welding electrodes or bits of shavings or parts of tools, that may have remained in the line after construction is completed.

As these pigs traverse the pipeline, their position is tracked by monitoring the emissions of an attached radio transmitter as well as by the indications of the above ground monitors (AGM) along the pipeline route that register the passage of the transmitters. If for some reason the pigs become ensnared in the line, the transmitters provide the location of the pig so that the appropriate measures can be taken to free it.

Figure 7 Foam Pigs and Gauge Pig with Lobed Plate

TRANSMISSION LINE CONDITION ASSESSMENT

Scope of Work

The scope of work developed by the energy company that was to be performed by the pipeline condition assessment provider consisted of:

1. One medium density foam pig cleaning run
2. One six cup cleaning pig run
3. One magnetic pig run
4. One gauge pig run
5. Two high resolution Geometry/Magnetic Flux runs
6. Provision and operation of temporary pig launchers and receivers

Due to the location of the transmission line in a densely populated urban area, the operations had challenges specific to the site. These challenges included:

1. Limited work areas and public activities adjacent to the in line tool launching site
2. Load and size restrictions on access roads to and at the launching and receiving sites
3. Limited access to the transmission line easement and right of way that traversed both residential and environmentally sensitive areas
4. Commercial and light industry activities close to the gas venting lines at the receiving site.

Performing the 24 Inch Diameter Gas Transmission Line Condition Assessment

The condition assessment was performed in April of 2011 and the final report was delivered in June of 2011. The first step in the process was to conduct a geographic position survey (GPS) along the route of the pipeline starting at the launch site (Figure 8) and ending at the receiver site (Figure 9). During the survey above ground monitoring stations (AGM) were located along the route at distances of approximately one half of a mile and at significant features, such as valves, and bends. Using the times at which the assessment tools pass by the monitors one can estimate their velocity and, if necessary, adjust the rate of flow and pressure of the gas propelling the instruments through pipeline route (Figure 10) so that it is within the correct range to obtain reliable instrument measurements.

Figure 8 Launch site

Figure 9 Receiver site

Figure 10 Pipeline route is blue line

Launching and Receiving the In Line Inspection Tools

Launching and receiving in line inspection tools follows the survey stage of the work. This requires a careful procedure to safely introduce and retrieve the tools while protecting the environment. Products within the pipeline should be treated as hazardous waste and handled as such. While the tools enter clean at the launch site, they emerge with all of the substances contained in the line and brought out during the cleaning and assessment procedure as they traverse the pipeline. The general procedures for launching and receiving tools in a pipeline containing gas are listed in the Figure 11. At the launch site for this project, close to buildings, slower line venting was required to reduce noise levels.

Launch Procedures Gas Service *

Starting condition: Trap is pressurized and full of gas. Mainline Trap Valve A, Main-line Bypass Valve B and Trap Kicker Valve C are open. Vent Valve(s) D, Drain Valve E and Equalization Valve F are closed.

A. Close Mainline Trap Valve A and Trap Kicker Valve C.

B. Open Vent Valve(s) D and Drain Valve E to vent trap to atmosphere.

C. When the trap is completely vented (zero psig) with Vent Valves D still in the open position, open the closure door and insert the pig so the first cup forms a tight fit in the reducer (point X). If a large and heavy inline inspection pig, it can be pulled into the trap by opening and using the pulling nozzles.

D. Close and secure the closure door. Open Equalization Valve F to allow permit pressure equalization in front of and behind the pig. If the pulling nozzles were used to load the pig, close them or replace and secure blind flanges. Purge air from trap through Vent Valves D and Drain Valve E by slowly opening Trap Kicker Valve C. When purge is completed, close Vent Valves D and Drain Valve E to allow pressure in trap to equalize with pipeline pressure. Then, close Valves C and F.

E. Open Mainline Trap Valve A, then Trap Kicker Valve C. The pig is now ready for launching.

F. Partially close Mainline Bypass Valve B. This will increase gas flow through Valve C and behind the pig. Continue to close Valve B until the pig moves out of the trap as signaled by the PIG-SIG scraper passage indicator.

G. When the pig leaves the trap and enters the main line, open Mainline Bypass Valve B fully.

Receiving Procedures Gas Service *

Pig receive systems are slightly different than launch systems. The most notable differences are length of piping, location of PIG-SIG Indicator and bypass piping.

As in launching a pig, receiving a pig should follow a described sequence to avoid similar problems. Therefore, the following procedures should be utilized to receive pigs. Starting Condition: Trap is empty at atmospheric pressure. Mainline Bypass Valve B, Vent Valve(s) D and Drain Valve E are open. Mainline Trap Valve A and Return Line Valve C are closed.

A. To purge air from the trap, close Drain Valve E. Slowly open Trap Kicker Valve C.

B. After purging, allow the trap pressure to equalize to pipeline pressure by closing Vent Valve(s) D, leaving Trap Bypass Valve C open.

C. With Trap Bypass Valve C still open, open Mainline Trap Valve A. The trap is now ready to receive the pig.

D. When the pig arrives, it may stop between the tee (point X) and the Mainline Trap Valve A.

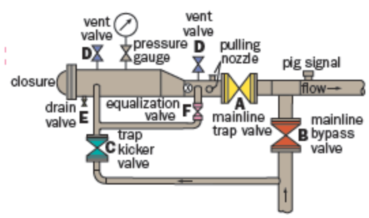
E. Partially close the Mainline Bypass Valve B. This will force the pig into the trap due to increasing flow behind the pig through the Mainline Trap Valve A and Return Line Valve C.

F. After the pig is in the trap, as indicated by the PIG-SIG pig passage indicator, open Mainline Bypass Valve B and close Mainline Trap Valve A and Return Line Valve C.

G. Open Drain Valve E and Vent Valves D to vent the trap to atmospheric pressure.

H. After the trap is vented (zero psig) and drained, with valves D and E open, open the closure door and remove the pig.

I. Close and secure the closure door.



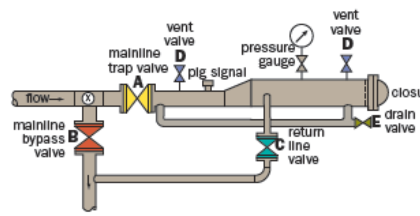


Figure 11 Launching and receiving procedures gas transmission line

Installation for Launching and Receiving In Line Inspection Tools

The installation of equipment at the launch and receiver sites (Figures 12, 13 and 14) required careful planning due to the weight of the launching equipment and the concern that the weight of the equipment mounted on low riding platform trucks might damage transmission lines under the access road. Wheel load calculations were required to confirm the underground pipes would not be overstressed. Due to restricted maneuver room, a high lifting capacity crane with a long boom needed to be positioned behind a building some distance from the launcher connection point to the pipeline (Figure 8). Once the launcher was installed, the crane required special permits to travel over local roads to the receiver site (Figure 10) to unload and install the receiver.



Figure 12 Launch connection point Figure 13 Unloading launcher Figure 14 Installing receiver

Pipe Line Tool Runs

Cleaning and Gauge Pigs

The day following the installation of the launching and receiving equipment adjustments were made and the medium density foam pig was sent through the line. When the foam pig was retrieved, no liquids were present and the pig had a slight coating of dust. The following day the cleaning brush pig with cups (Figure 15) and the gauge pig were propelled through the pipeline at 300 psi pressure. Each pig brought up some small amount of fluid and some pieces of a tape measure and welding electrode left in the pipe since the time it was built. The gauge pig had a slight deflection on one of the lobes, but was within tolerance of 22 inches for the DEF tool run.

For cleaning and gauge pig runs, personnel tracked the pigs along the route in order to control pig position and velocities, which averaged about 3.5 miles per hour. Knowing the position of the pigs and tools at all times is important if for some reason they become ensnared in the line and steps need to be taken to reinitiate movement or remove them.

Normally two to three technicians are employed for tracking. At the time of launch, the trackers are positioned sequentially at the first AGM stations installed prior to the runs. As the pig or tool passes the particular point, the corresponding technician moves forward along the transmission line to the next unmanned station. Technicians need to move along existing roads and streets in urban areas. In the particular case described herein, traffic congestion and railroad and electrical transmission line right of ways created limited direct access which, along with green areas and parks (Figure 10) made the task difficult for the technicians.

DEF and MFL Pig Runs

The DEF tool was run the day following the cleaning pig runs (Figure 16). The run lasted about four hours and run data was confirmed to be good by the technician. The following day the MFL tool was run within the same approximate run time as the previous day (Figure 17).


Figure 15 Brush cleaning pig after completing run Figure 16 DEF tool extracted from receiver

Liquids produced from the runs was collected, treated and removed from the site by the environmental remediation company contracted by the pipeline company, which also washed the ILI tools (Figure 18). The following day launchers and receivers were disassembled and loaded on trucks for return to their point of origin.

Figure 17 MFL Tool Extracted from Receiver

Figure 18 Environmental Containment at Receiver

Preliminary Report Results

In May, a preliminary report of the DEF and MFL runs was submitted to the energy company. The report indicated, among others, the number of welds encountered, valves, fittings "Tees", flanges and deformations. The report confirmed that the MFL run detected the metal losses measured were such that, based on the criteria of ASME B31.G: Modified, it was safe to operate the pipe at the current Maximum Allowable Operating Pressure of 596 psi, above red line in Figure 19. Furthermore, the report showed that the DEF tool run found minor deformations and no ovalities or expansions (Figure 20) and therefore no immediate repairs were required. The final report that was delivered provided more detailed information for the energy company.

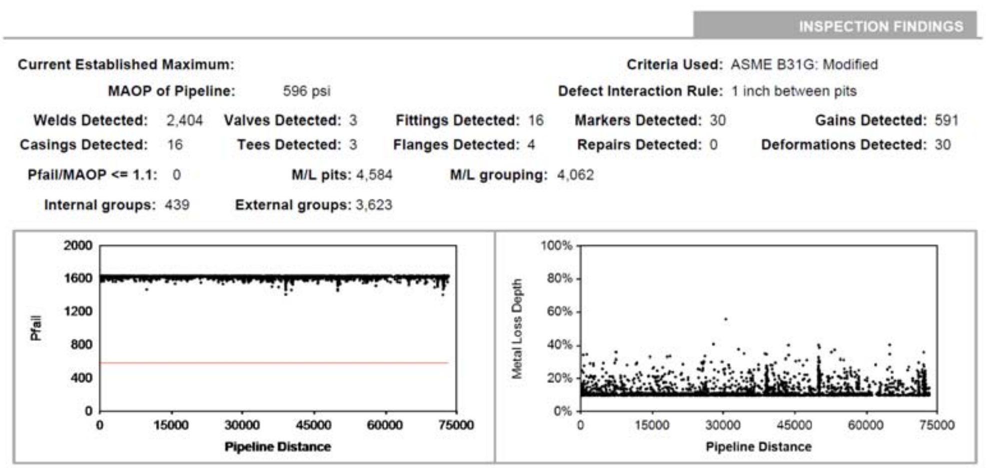


Figure 19 Pipeline features and wall thickness detected along pipeline

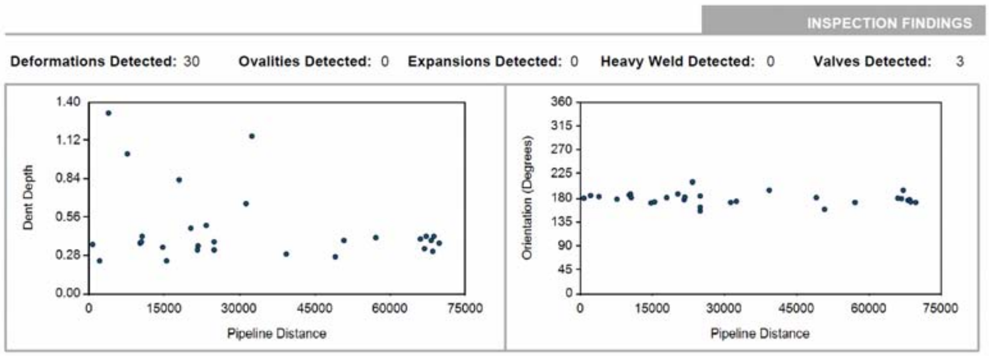


Figure 20 Pipeline features and wall thickness detected along pipeline

CONCLUSION

A condition assessment of a 13 mile long, 24-inch gas transmission line, located in a densely populated suburban area of a large Midwest City was accomplished efficiently without incident by using progressive pigging methods and the appropriate in line inspection tools. The assessment indicated that, based on the criteria of ASME B31.G: Modified, the line was operationally safe and that no immediate repairs were required. The information gathered will serve as a basis for projecting pipeline maintenance programs.

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Influences on the Rate of Pressure Drop in Automatic Line Break Control Valves on a Natural Gas Pipeline

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Abstract
When a natural gas pipeline ruptures, the adjacent upstream and downstream automatic line break control valves (ALCVs) should close quickly to prevent significant as leakage. The rate of pressure drop (ROD) setting and ROD duration time are the important parameters that determine whether an ALCV closes in time. This study used transient hydraulic software to evaluate the effect of parameters such as flow rate, operating pressure, distance from a rupture to a valve, and rupture diameter on maximum ROD values of ALCVs installed on the second Shannxi-Beijing gas pipeline. In addition to analyzing the effects of pipeline ruptures, the study also evaluated the effects of valve closing. The study found that pipeline flow rate has little effect on the maximum ROD value caused by a rupture. Operating pressure, distance from a rupture to a valve, and rupture diameter have large effects on the maximum ROD value caused by a rupture. Flow rate and operating pressure have the greatest impacts on the maximum ROD value caused by a valve shutdown. In addition, the greater the ROD duration time, the smaller should be the allowable maximum ROD setting.
Keywords: Natural gas pipeline; Automatic line break control valve; Rate of pressure drop.

1. Introduction

When a natural gas pipeline ruptures, the adjacent upstream and downstream automatic line break control valves (ALCVs) should close quickly to prevent significant gas leakage and the resulting economic loss, as well as to minimize the likelihood that the rupture will cause a catastrophic accident. The rate of pressure drop (ROD) setting value and ROD duration time are the important parameters that determine whether an ALCV closes in time. Previously, because calculation of ROD setting values for ALCVs is complex, especially when operating conditions along a pipeline vary, ROD setting values have been usually adopted based on domestic or foreign experience, or based on a crude estimate of the pipeline steady flow (Wang *et al*., 2004; Wang *et al*., 2013). In fact, using a unified ROD setting value to protect different pipelines, or even a single pipeline with varied operating conditions, has a great deal of uncertainty due to unsteady gas flow in a pipeline under rupture conditions (Phan *et al*., 2012). Thus, ALCVs set according to a convention that fails to account for site-specific influences may malfunction under rupture situations, either by failing to close quickly enough or closing unnecessarily. Obviously, a malfunctioning ALCV cannot isolate a pipe section rapidly in the event of an accident, so as to protect the rest of the gas pipeline and reduce the economic losses. The correctness of an ROD setting value directly relates to the accuracy and timeliness of the ALCV action. Thus, the analysis of how factors influence the maximum ROD value of ALCVs under various operating conditions has great significance.

The second Shaanxi-Beijing gas pipeline is 939.6 km long, with an outside diameter of 1016 mm and a design operating pressure of 10 MPa. The flow rate is 17 billion cubic meters per year. The spacing of automatic line break control valves along the pipeline is shown in Fig.1, but is typically 10–30 km. The longest valve spacing is 31.4 km, and the shortest spacing is 5.2 km.

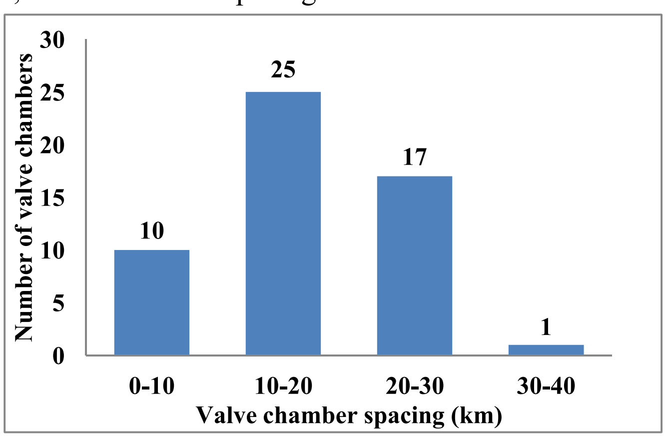

Fig. 1 ALCV chamber spacing for the second Shaanxi-Beijing gas pipeline

2. ROD Calculation Method

The LineGuard model of a Shafer gas-hydraulic actuator records pressure values in an ALCV chamber once every 5 s, as shown in Fig. 2.

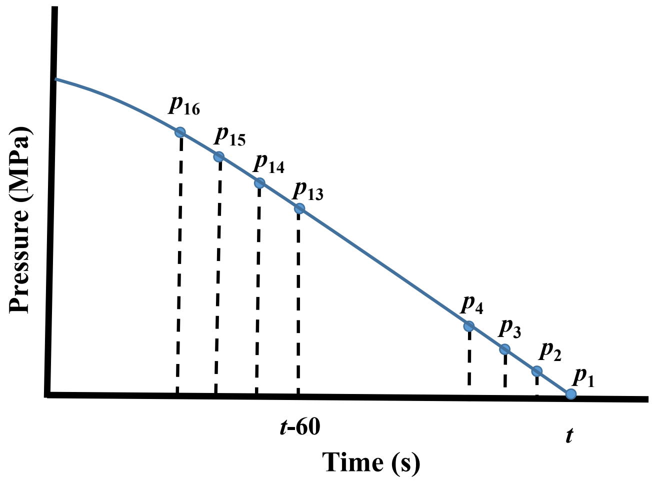

Fig. 2 Schematic of pressure data sampling

According to Fig. 2, the average pressure value at time t s $P_{avg,t}$ is calculated using Eq. 1:

$$P_{avg,t} = (p_1 + p_2 + p_3 + p_4)/4 \tag{1}$$

where, p_1, p_2, p_3 and p_4 are sequential pressure measurements at 5 s intervals, MPa.

Likewise, the average pressure value at time t–60 s $P_{avg,t-60}$ is calculated using Eq. 2:

$$P_{avg,t-60} = (p_{13} + p_{14} + p_{15} + p_{16})/4 \tag{2}$$

where, p_{13}, p_{14}, p_{15} and p_{16} are sequential pressure measurements at 5 s intervals, MPa.

The rate of pressure drop (*ROD*, MPa/min) value at t s is calculated using Eq. 3:

$$ROD = P_{avg,t-60} - P_{avg,t} \tag{3}$$

Transient hydraulic simulation software PipelineStudio 3.2.7.5 (Energy Solutions International, Houston, Texas) capable of calculating instantaneous pressure was used in this analysis. In order to accurately model the line break control valve calculation, pressure simulation results were exported, and the ROD values were calculated outside of the simulation software and further analyzed.

An example simulation for the second Shaanxi-Beijing gas pipeline is shown in Fig. 3. The simulated flow rate is $2800\times10^4 m^3/d$; the operating pressure at the valve chamber is 9.5–8.0 MPa; the rupture is located one-third of the distance from the upstream valve chamber to the downstream valve chamber; the rupture diameter is 400 mm; and the rupture diameter enlargement time is 5 s, i.e. the rupture diameter extended from 0 to 400 mm in 5 s. The ROD values versus time at both the upstream and downstream valve chambers are shown in Fig. 3.

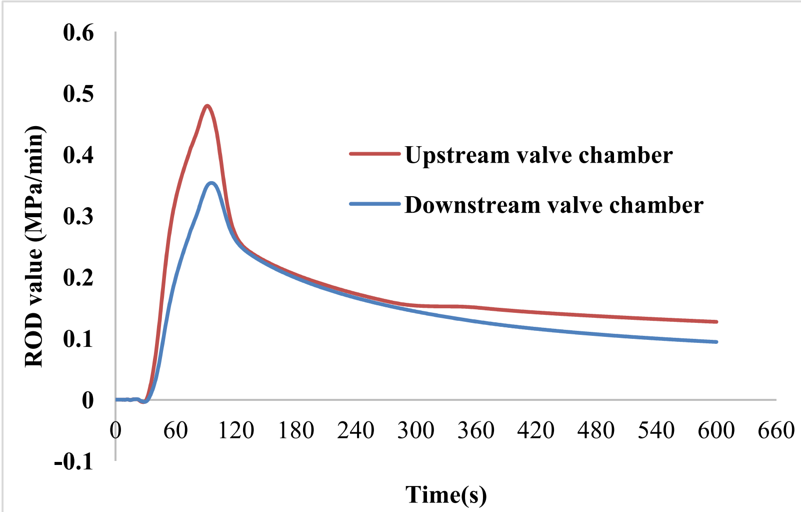

Fig. 3 ROD value versus time at upstream and downstream valve chambers

From Fig. 3, if the ROD duration time is 120 s, the maximum ROD value at the upstream valve chamber is 0.22 MPa/min; thus, a setting of 0.22 MPa/min and lower over 120 s would detect the rupture. The maximum ROD value at the downstream valve chamber is 0.17 MPa/min; therefore, a setting of 0.17 MPa/min and lower over 120 s would detect the rupture.

3. ROD Setting Value of an Automatic Line Break Control Valve

To determine the ROD setting of an ALCV, many factors must be considered, which can be divided into two groups:

(1) Accident-related conditions that must be detected, such as a pipeline rupture. In order to detect such conditions, the ROD setting value must be lower than the maximum ROD value that would occur over a pre-set period of time as a result of a rupture.

(2) Normal conditions that should be ignored, such as pressure changes due to a valve closing or to a compressor stopping. For an ALCV to ignore normal conditions (i.e., not actuate), the ROD setting value must be higher than the maximum ROD value that would occur over a pre-set period of time as a result of the events.

3.1 Pipeline rupture conditions that need to be detected

A variety of pipeline rupture parameters were examined:

(1) Flow rate: 2800, 3400, 4400, 4850×$10^4$$m^3$/d;

(2) Operating pressure range: 9.5–8.0, 8.0–6.5, 6.5–5.0 MPa;

(3) Rupture position: 1/3, 1/2, 2/3 of the distance along a pipe section from upstream valve chamber to downstream valve chamber;

(4) Rupture diameter: 150, 300, 400, 500 mm;

(5) Rupture diameter enlargement time: 5 s, 300 s.

Permutations of the above parameters in combination resulted in 288 different rupture scenarios that were analyzed using transient simulation. The simulation results were obtained after processing and statistical analysis. For example, when the rupture diameter enlargement time is 5 s and the ROD duration time is 120 s, the maximum ROD values arising from the various rupture conditions in the upstream valve chamber are shown in Fig. 4 to Fig. 6. Likewise, the maximum ROD values at the downstream valve chamber are shown in Fig. 7 to Fig. 9.

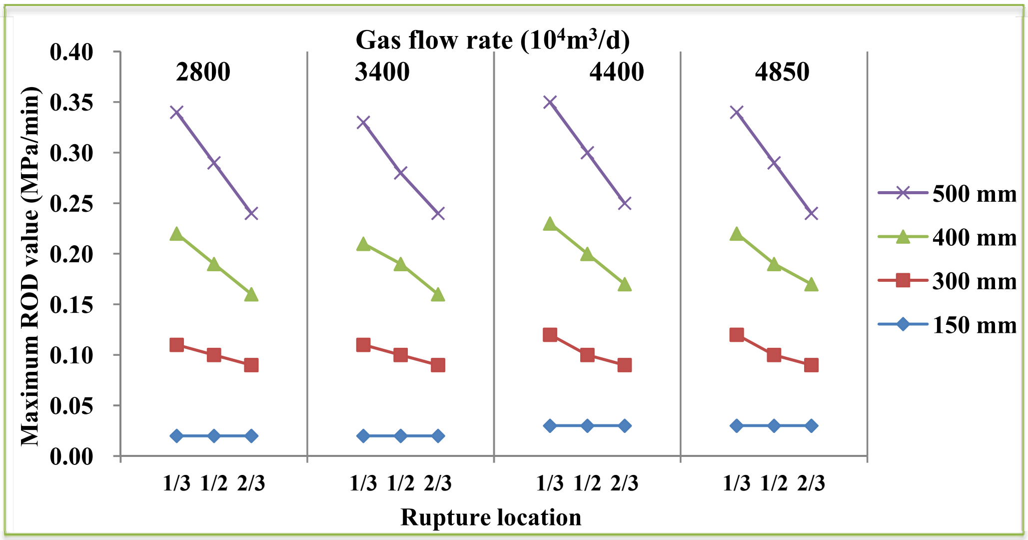

Fig. 4 Maximum simulated ROD values in upstream valve chambers arising from different rupture conditions (operating pressure 9.5–8.0 MPa)

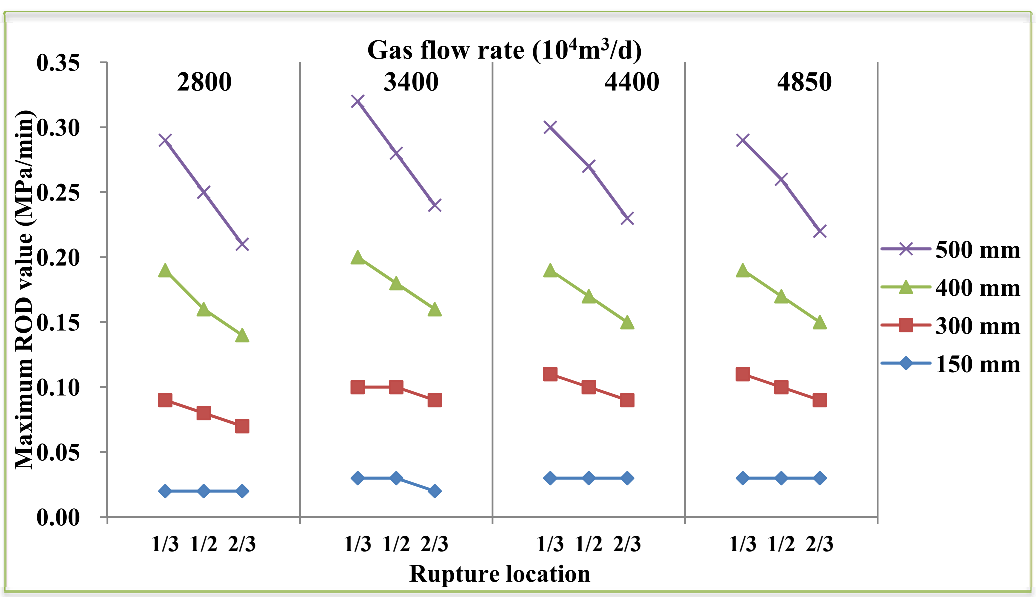

Fig. 5 Maximum simulated ROD values in upstream valve chambers arising from different rupture conditions (operating pressure 8.0–6.5 MPa)

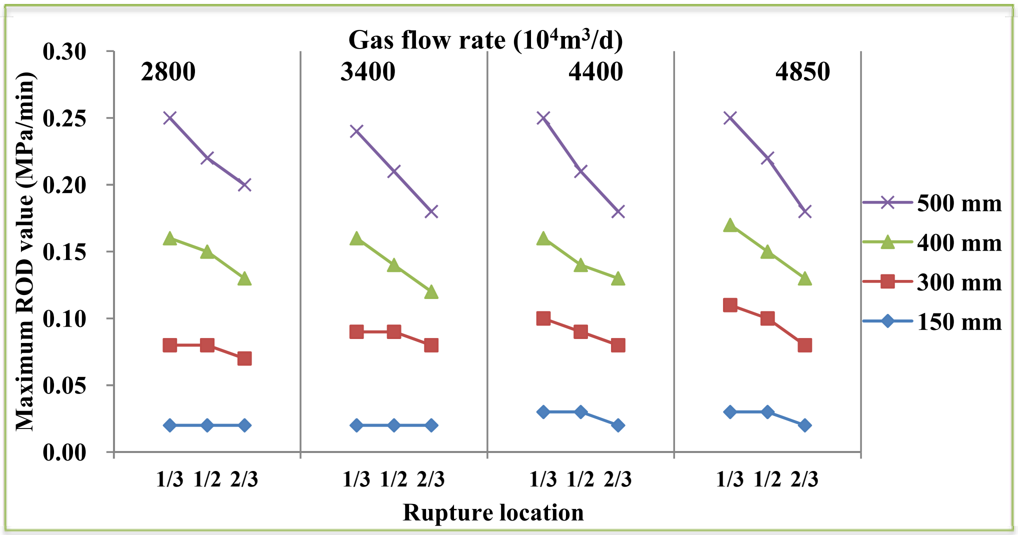

Fig. 6 Maximum simulated ROD values in upstream valve chambers arising from different rupture conditions (operating pressure 6.5–5.0 MPa)

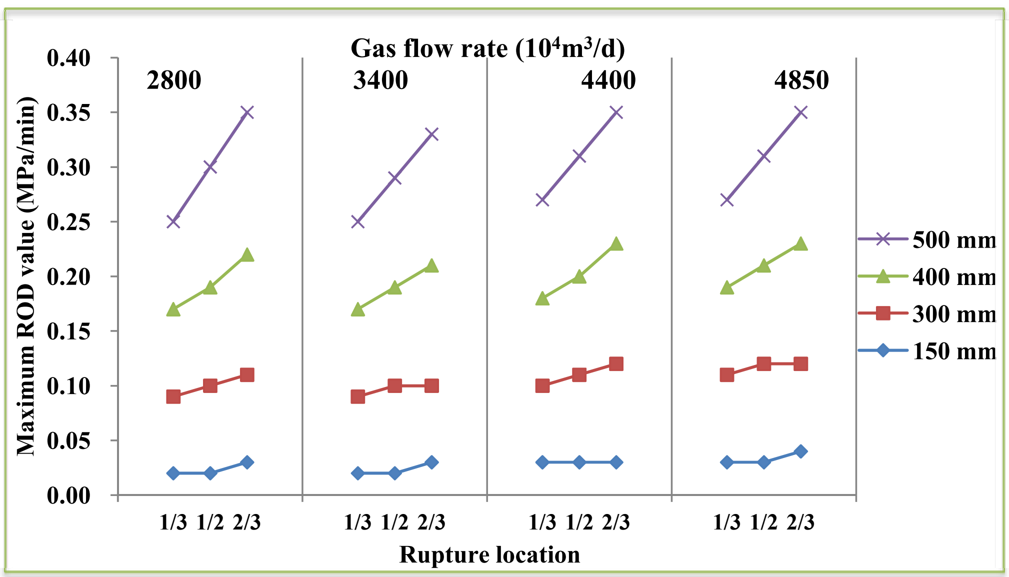

Fig. 7 Maximum simulated ROD values in downstream valve chambers arising from different rupture conditions (operating pressure 9.5–8.0 MPa)

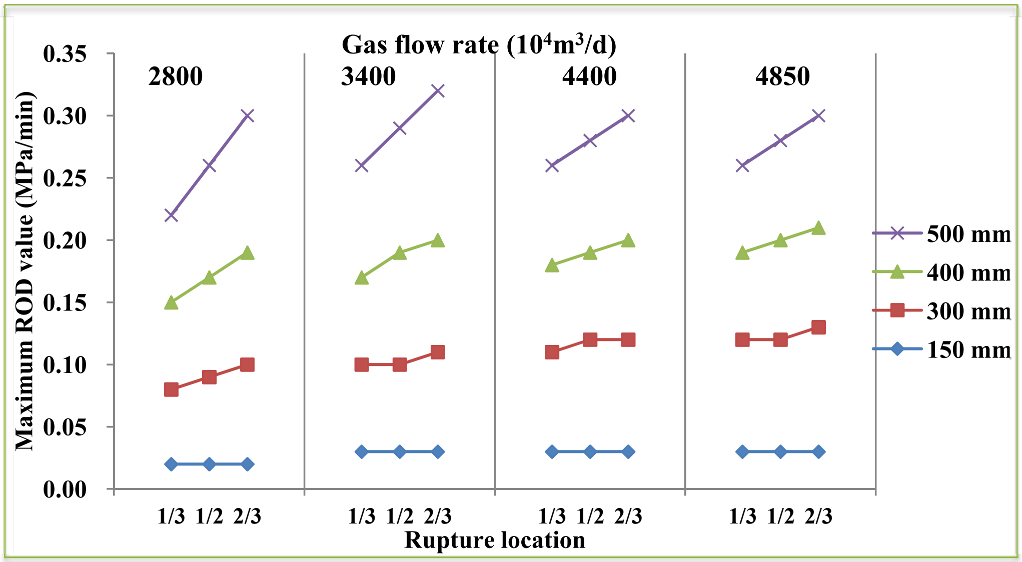

Fig. 8 Maximum simulated ROD values in downstream valve chambers arising from different rupture conditions (operating pressure 8.0–6.5 MPa)

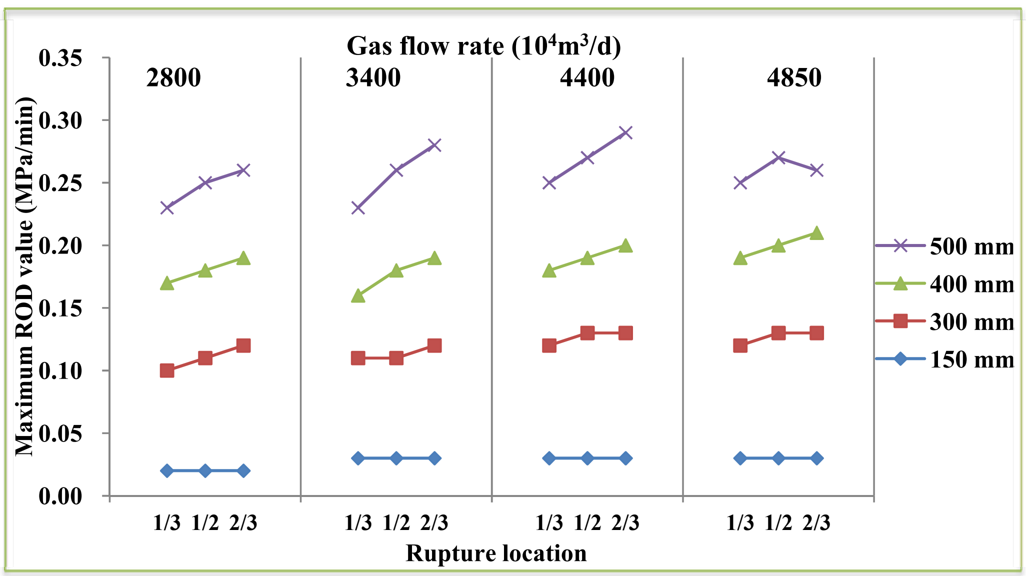

Fig. 9 Maximum simulated ROD values in downstream valve chambers arising from different rupture conditions (operating pressure 6.5–5.0 MPa)

From Figs. 4–9, it is not difficult to see that rupture diameter has a great impact on the resulting maximum ROD values at both the upstream and downstream valve chambers. For example, at the operating pressure range of 9.5–8.0 MPa, the increase in rupture diameter from 150 mm to 500 mm caused the maximum ROD value in the upstream valve chamber (Fig. 4) to increase from 0.02 MPa/min to 0.34 MPa/min.

The rupture location also has an influence on the maximum ROD value that occurs at both the upstream and downstream valve chambers, but the severity of the impact is associated with the rupture diameter. In general, the nearer a rupture location to a valve chamber is, the larger will be the resulting maximum ROD value;

and, as the rupture diameter increases, the effect on maximum ROD is more obvious.

Reducing the pipeline operating pressure causes the rupture-related maximum ROD at the upstream and downstream valve chambers to decrease. When the pipeline operating pressure is in the 9.5–8.0 MPa range, and the rupture diameter increases from 150 to 500 mm, the maximum ROD value in the upstream valve chamber increases from 0.02 to 0.34 MPa/min. However, when the pipeline operating pressure is in the 6.5–5.0 MPa range, and rupture diameter increases from 150 to 500 mm, the maximum ROD value in the upstream valve chamber increases from 0.02 to only 0.25 MPa/min.

Under the rupture conditions investigated, gas flow rate has little effect on the maximum ROD value that occurs at upstream and downstream valve chambers.

Compared with the downstream valve chamber, it is more difficult for the upstream valve chamber to detect a pipeline rupture because the maximum ROD value observed there is smaller.

The current ROD setting of the second Shaanxi-Beijing gas pipeline is 0.15 MPa/min over 120 s. Thus, according to the analyses portrayed in Figs. 4–9, this set value can only detect a rupture with diameter of 500 mm (and some 400 mm ruptures). Ruptures with diameters of 150 mm and 300 mm cannot be detected.

Simulated maximum ROD values when the gas flow rate in the second Shaanxi-Beijing gas pipeline is $3400\times10^4 m^3/d$ are shown in Fig. 10. For these simulations, the operating pressure is between 6.5–8.0 MPa and the rupture diameter is 400 mm. The relationship between the maximum ROD value and ROD duration time is shown in Fig. 10.

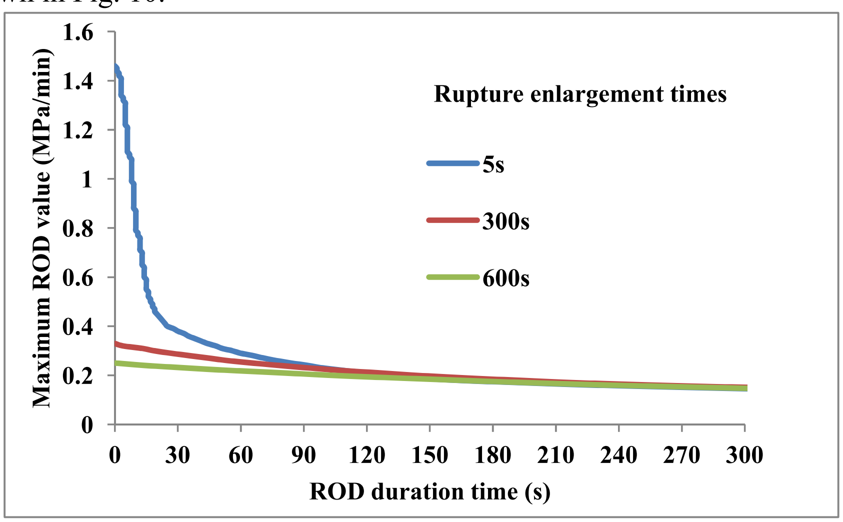

Fig. 10 The relationship between maximum ROD value and ROD duration time

From Fig. 10, when ruptures occur, the longer the rupture diameter enlargement time, the lower the ROD setting value can be, but this relationship is relatively insensitive.

If the equipment reliability is high, the ROD duration time can be reduced to 90 s, but the ROD setting value should be increased accordingly. If the device reliability is not high, the ROD duration time can be increased to 180 s or even 240 s, and the corresponding ROD setting value should be decreased accordingly, as indicated in Fig. 10.

3.2 Valve closing conditions that should be excluded

Pressure changes resulting from the closing of automatic line break valves of the second Shaanxi-Beijing gas pipeline were also simulated using PipelineStudio. When the upstream block valve closes, the predicted maximum ROD values (over a 120 s duration) at a downstream valve chamber are shown in Fig. 11 to Fig. 14. “8.0-9.5 (2800)” means the operating pressure is in the 8.0-9.5 MPa range and the gas flow rate is 2800×10^4 m^3/d.

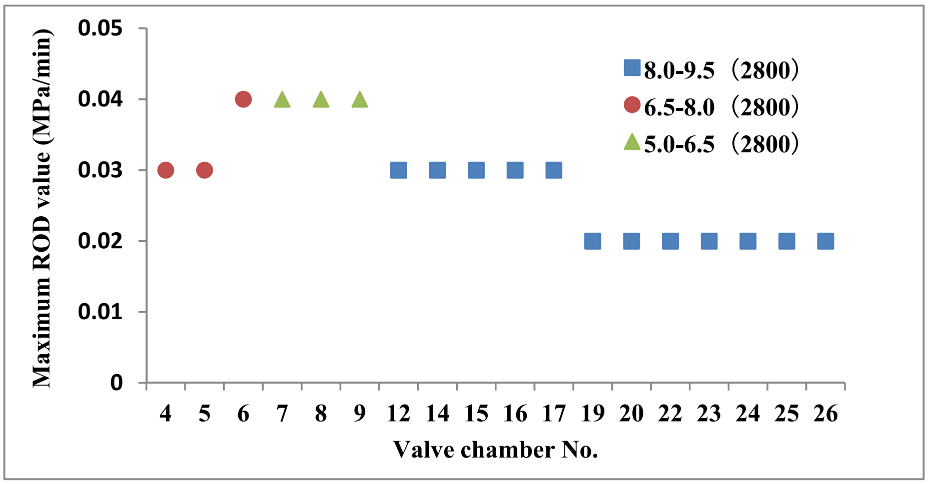

Fig. 11 Maximum ROD values over 120 s at downstream valve chambers resulting from closure of an adjacent upstream valve (gas flow rate 2800×10^4 m^3/d)

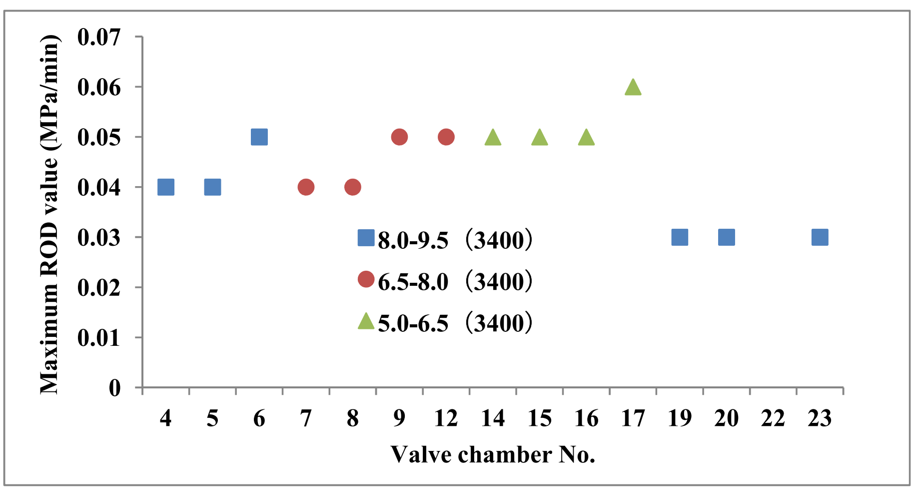

Fig. 12 Maximum ROD values over 120 s at downstream valve chambers resulting from closure of an adjacent upstream valve (gas flow rate 3400×10^4 m^3/d)

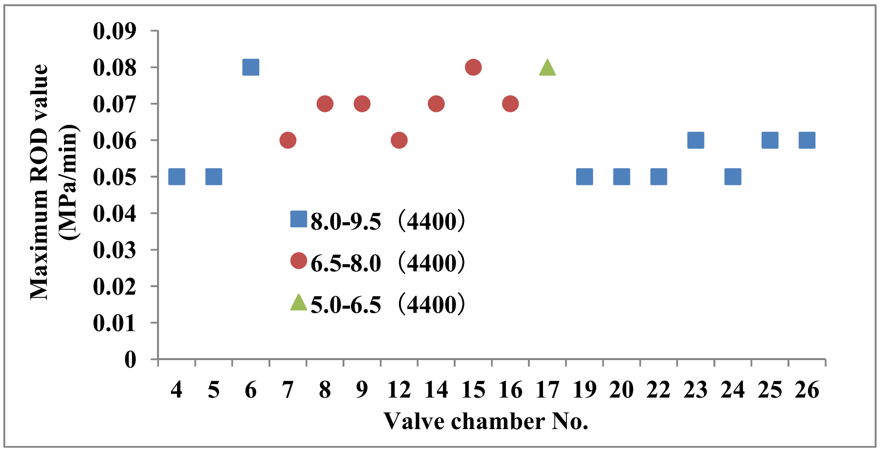

Fig. 13 Maximum ROD values over 120 s at downstream valve chambers resulting from closure of an adjacent upstream valve (gas flow rate 4400$\times 10^4$ m^3/d)

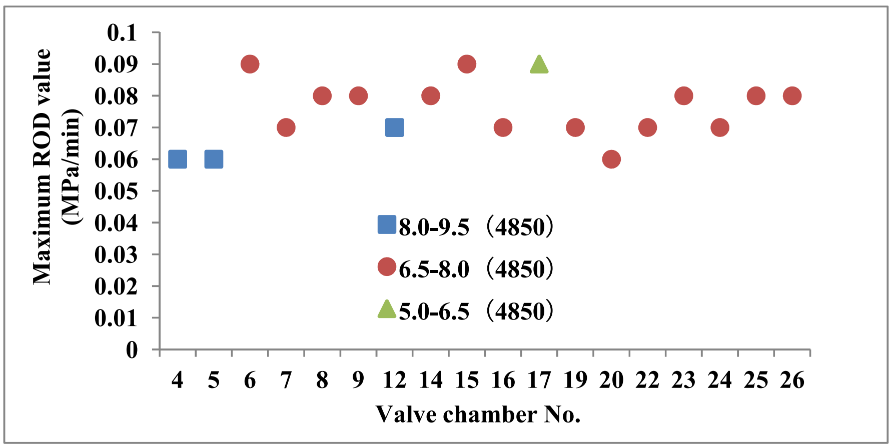

Fig. 14 Maximum ROD values over 120 s at downstream valve chambers resulting from closure of an adjacent upstream valve (gas flow rate 4850$\times 10^4$ m^3/d)

Fig. 11 to Fig. 14 indicate that:

(1) With an increase of the pipeline flow rate, when an upstream valve closes, the maximum ROD values over 120 s at the adjacent downstream valve chamber will increase. The maximum ROD value over 120 s at a downstream valve chamber is 0.09 MPa/min.

(2) With a decrease of the pipeline operating pressure, when an upstream valve closes, the maximum ROD values over 120 s at the adjacent downstream valve chamber will increase.

3.3 Summary

The actual ROD setting value for all ALCVs on the second Shaanxi-Beijing gas pipeline is 0.15 MPa/min over 120 s; considering a safety margin, this value is relatively reasonable. Simulations show that at this ROD setting value, the closure of an upstream valve will not cause a downstream valve to close. Of the rupture scenarios evaluated, the ROD setting of 0.15 MPa/min over 120 s will detect only a

500 mm rupture under all conditions, and a 400 mm rupture under limited conditions. In order to further refine the pipeline management, it is recommended that instead of using a single ROD setting value for all ALCVs on the pipeline, the ROD setting for each automatic line break control valve should be determined individually considering different operating conditions.

4. Conclusions

Simulations of scenarios that could cause rapid pressure changes in the second Shaanxi-Beijing gas pipeline support the following conclusions.

(1) Automatic line break control valves can be used to detect only pipeline ruptures that are at least half the pipeline diameter in size. ALCVs cannot be used to detect small-diameter perforations, such as those caused by mechanical damage and corrosion.

(2) The diameter of a rupture has a great impact on the resulting maximum ROD value at both the upstream and downstream valve chambers. As rupture diameter increases, the maximum ROD value in the adjacent valve chamber increases.

(3) The location of a rupture relative to an ALCV has some influence on the maximum ROD values at upstream and downstream valve chambers, but the severity of this impact is associated with rupture diameter. The closer a rupture is to a valve chamber, the larger will be the maximum ROD value at the valve; as the rupture diameter increases, this relationship becomes more obvious.

(4) When ruptures occur, the pipeline operating pressure has a great impact on the maximum ROD values at upstream and downstream valve chambers. As the pipeline operating pressure decreases, the maximum ROD values at both the upstream and downstream valve chambers decrease.

(5) When a rupture occurs, gas flow rate has little effect on the maximum ROD value at upstream and downstream valve chambers.

(6) Ruptures are comparatively more difficult to detect by an adjacent upstream ALCV than by an adjacent downstream ALCV.

(7) Under valve closing conditions, the resulting maximum ROD value at a downstream valve chamber increases either as the gas flow rate increases or as the operating pressure decreases.

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More Precise Hydro-Static Test Evaluation of High Pressure Petroleum Pipelines Using Automated Data Collection Techniques

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Abstract

During a cross-country steel pipeline hydro-static test, pressure changes throughout the test, as a result of temperature fluctuations of the steel & water, air in the system or the presence of a leak in the system. Petroleum releases can occur if the thermodynamics of the test mask a pipe or construction related defect. Using precise measurement techniques, it is possible to evaluate the effects of temperature fluctuations in the system, and greatly improve the ability to identify a small leak during the hydro-static test. When the pipeline being tested is buried for many miles, it is critical to be able to identify leaks without visual inspection. This technical paper will present a brief description of common hydro-static testing measurement equipment in use today. Second, a description of the digital instrument and automated data collection measurement system recently developed by the authors will be presented. Third, a brief description of hydro-test thermodynamics will be given. Then the process of conducting the leak test portion of the hydro-static test will be explained. Finally, a comparison of the relative error between the common hydro-static testing measurement equipment in use today and the digital instrument and automated data collection measurement system recently developed will be presented.

Introduction

The purpose of a pipeline hydro-static test is to meet the hydro-static test objectives.

- Detect and eliminate anomalies in a pipeline segment in new pipeline construction or in existing pipeline systems.
- Establish the maximum operating pressure limit of a pipeline segment in new pipeline construction.
- Verify the integrity of an existing pipeline to ensure a leak is not present in an existing pipeline system.

A pipeline hydro-static test may be performed to meet regulatory requirements or it may also be performed to meet engineering design code requirements. Both the regulatory requirements and the engineering design codes require a strength test and a leak test component to hydro-static tests. Both the strength test and leak test are achieved during the one hydro-static test period.

A strength test proves the pressure carrying capacity of the pipe. The common hydro-static testing parameter is to test at a pressure that is 125% of the intended operating pressure. If the intended operting pressure is 1,440 PSI, then the test would be at a minimum of 1.25*1,440 PSI = 1,800 PSI. As long as the entire hydro-static test is conducted above this minimum pressure, the strength test requirement of the hydro-static test is met, for the stated operating pressure.

A leak test ensures the test was completed without leakage. A simple way to achieve this result is to have an entirely above ground hydro-static test and to visually inspect for leaks during the test. However, on cross-county pipelines, most of the pipe is buried during a hydro-static test. In situations where a leak test is required with buried piping, careful measurement practices allow the engineer to infer what is happening below ground, unseen, which enables the engineer to certify the successful completion of a leak test. A hydro-static test is considered successful once both the strength test and leak test requirements are met. Using the methods described in this paper, along with controlling the five basic assumptions of the leak test calculations described below, it is possible even for a relatively inexperienced engineer to conduct and certify the successful completion of leak tests with repeatable results and a justifiable mathematical basis.

Common Hydro-Static Testing Measurement Practices

Measurement equipment that may be encountered during a hydro-static test includes:

- Deadweight pressure tester
- Digital pressure gauge
- Analog circular pressure chart
- Analog circular temperature chart

A deadweight pressure tester is an analog device that operates on the principle of balancing a known mass on a known area of a plate to determine pressure. These devices are operated by a human operator. Accuracies down to 0.1% of the pressure reading are achievable. This technology was among the first available to accurately record pressure during a hydro-static test and is regularly used today.

Digital pressure gauges are also fairly common in the hydro-static testing industry. *Crystal* is a common brand of digital pressure gauge. The accuracy of digital pressure gauges also depends on the manufacturer, but a particular model of *Crystal* gauge is accurate to 0.1% of the pressure reading. These types of gauges are convenient because they remove the human error associated with operating a deadweight pressure tester and in addition they have the capability of electronically storing data.

An analog circular pressure chart is a device that continually records pressure during a test. The most prevalent brand name is *Barton*. The accuracy of these devices depend on the manufacturer but one particular model of *Barton* chart is accurate to 1% of the full scale pressure reading.

An analog circular temperature chart is a device that continually records temperature during a test. Sometimes a dual pressure/temperature circular chart can be used. The accuracy of these devices, too, depend on the manufacturer but one particular model of *Barton* chart is accurate to 1% of the full scale temperature reading.

Some combination of the pressure and temperature sensing instruments listed above are placed around the hydro-static testing system during the hydro-static test in order to conduct the strength test and leak test portions of the hydro-static test. Unless careful attention is given to the placement of temperature measurements, it is not possible to conduct a leak test on a buried pipeline precisely enough to prevent possible leaks from going undiscovered during the test.

Digital Instrument & Automated Data Collection System

In order to more precisely conduct leak tests during hydro-static testing, a digital instrument and automated data collection system was developed by the authors to assist the engineer in the evaluation of leak tests. The testing protocol described below gives the engineer enough information to conduct and evaluate a leak test, as well as a strength test.

The major differences between this measurement system and conventional hydro-static test measuring equipment are:

- Numerous buried temperature measurements are attached to the buried pipeline to better approximate the overall temperature of the test medium.
- Continuous real-time monitoring of the data allows the engineer to make prompt decisions regarding how the test is proceeding and if any remedial action is required for the pipeline system to pass a leak test.

Temperature measurement is taken at a sufficient number of points to get an adequate representation of above ground and below ground pipeline temperature. This includes a number of below ground temperature measurements. The more temperature measurements that are available, the more accurate the physical model of the pipeline temperature profile will be.

A pressure measurement kit contains the following:

- *Crystal* gauge for primary pressure measurement
- Pressure transmitter (manufactured by Aultrol) for secondary pressure measurement
- Radio transmitter (manufactured by Signal-Fire)
- Base station (manufactured by Signal-Fire)

A digital *Crystal* gauge is used as the primary pressure sensing element. This device is very accurate and the results are repeatable.

A digital *Aultrol* pressure transmitter is used as a secondary pressure sensing element. Should the primary pressure sensing element fail, the secondary pressure element can be used to evaluate the test.

The pressure transmitter is used to record pressure during the test to serve as the secondary pressure measurement. The data is communicated via the radio transmitter to the base station that is plugged into a computer to download the data during the test in real time.

A temperature measurement kit contains the following:

- Temperature transmitters (manufactured by Aultrol)
- Radio transmitter (manufactured by Signal-Fire)

The temperature transmitters are used to record temperature during the test. Resistance temperature detectors (RTD's) are installed both below ground and above ground. The *Aultrol* temperature transmitters have an error of 0.1% of full temperature scale. The RTDs that are installed during the test are provided by Smart Sensors Inc. These RTDs have an error of 0.3 °F + 0.5% of temperature reading.

During a hydro-static test, all of the temperature and pressure measurements are recorded and transmitted remotely to the engineer. Each automation kit is powered by a solar panel with a battery back-up system to increase reliability in the field during hydro-static tests. The range of communication on the radio transmitters is three miles with good line of sight. Each radio transmitter also acts as a repeater for the other radio transmitters in the area to communicate the information to the engineer in real time during the test.

The automation kits eliminate the human error associated with the manual reading and recording of data. This continuous real-time monitoring of data allows the engineer to make real-time decisions during the hydro-static test to either stop the test and investigate for possible leaks or some other cause of unexplained pressure change. It also allows the engineer to immediately certify the completion of a successful hydro-static test once the test duration is complete. This can often save several hours of contractor time if the information would otherwise have to be collected and communicated to a home office engineer.

Thermodynamics of a Pipeline Pressure Test

A complete derivation of the mathematical equations behind the relationship between temperature and pressure for pipeline systems during a hydro-test is beyond the scope of this technical paper. However, the basic premise is that in a closed thermodynamic system, if heat is applied and the temperature of the system rises, that temperature rise will increase the volume of space that the steel and the water both try to occupy. The volume change that occurs in both the water and the steel causes a corresponding pressure change because the buried pipeline system is constrained against completely free movement by the Earth. In a closed system, when temperature increases, pressure will increase, and when temperature decreases, pressure will decrease.

Leak Test Portion of a Hydro-Static Test

If the entire hydro-static test is above ground, a visual inspection for leaks is the preferred method for detecting leaks. If some of the pipeline is below ground, then the engineer cannot see what is happening for a portion of the test. Generally, once a hydro-static test starts, pressure will fluctuate. If the strength test portion of the hydro-static test is successful, then some questions the engineer is faced with at the conclusion of the test are:

- Was the observed pressure change during the test due to a leak?
- Was the observed pressure change during the test due to temperature change of the test medium?
- Was the observed pressure change during the test due to excess air in the pipeline system?
- Was the observed pressure change during the test due to something else that is unknown?

The engineer is faced with the difficult task of determining whether or not a successful leak test has been performed. Pressure measurement alone is not enough to determine if a successful leak test has been performed.

The most common cause of a pressure change during a hydro-static test, if it is not being caused by a pipeline leak, is the variation of temperature of the test medium. The above ground temperatures change dramatically following the day/night cycle and ambient temperature changes. The below ground temperatures change less dramatically, however, their small changes are even more important than the above ground temperature changes, since, usually the vast majority of the pipeline is buried.

As temperature rises or falls during a test, corresponding pressure changes will occur in the system. This relationship requires that the measurement of pressure and temperature be precise and representative of the test section in order to explain the pressure variations that are observed during the test.

Test pressures can be measured and determined for the entire pipeline hydro-static test with a high degree of certainty. The use of an elevation profile along with a single pressure measurement location provides an accurate representation of the test pressures along the entire pipeline. Temperature variations in the pipeline system are caused by varying burial depths and multiple ground environments which have different heat transfer properties. The precise temperature of the test medium is not known. It is not practical to measure every location where a temperature difference exists. However, the number and location of temperature measurement points should be evaluated and should also adequately describe the test medium temperature profile. It is also important to allow the test medium temperature in the pipeline to stabilize before pressure testing begins. A degree of uncertainty will exist due to the number and accuracy of the instrumentation and this uncertainty should be taken into account when establishing acceptance criteria. (API 1110, 2013).

The hydro-static leak test calculations give the engineer the ability to predict pressure change based on temperature measurements during a test. However, there are limitations of the leak test calculations and the engineer must be aware of these limitations and the assumptions in the calculations in order to prevent “passing” a hydro-static test for a pipeline that was leaking during the test, unseen. The pipeline system during a hydro-static test is a complex, transient heat transfer scenario during the test where temperature and pressure are constantly changing throughout the test and the engineer is asked to certify the test without knowing the entire continuum test medium temperature.

The five basic assumptions that go into the leak test calculations are as follows:

- No air is trapped within the pipeline system.
- The water is pure and its properties are uniform.
- The soil restraint boundary conditions are known.
- The pipeline wall thickness is constant.
- The measured temperature indicates the entire test medium temperature.

Strictly speaking, none of the above five assumptions are perfectly true. However, the better these assumptions are managed, the more accurately the leak test calculations can explain the pressure change in a pipeline system during a hydro-static test. If the above five basic assumptions are controlled to a reasonable level, based on the certifying engineer’s past experiences as well as good engineering practice, then it is possible to confidently certify the successful conduct of a leak test.

Working to remove as much air from the system as possible and to allow the test medium temperature to stabilize before initiating the pressure test greatly improve the quality of data used to evaluate the leak test.

The single biggest challenge with the leak test evaluation is to get the measured temperatures to indicate the entire pipeline test medium temperature, with limited installation of temperature devices. The engineer must decide how many temperature measurements are sufficient to address this problem. The premise is to gather enough temperature measurements to create an accurate model of the test medium temperature in order to explain how much the pressure should change during a test.

The authors have had much success by placing a temperature measurement every mile or two, making sure to capture both above ground and below ground measurement locations that are typical for that pipeline system in order to evaluate leak tests. Additionally, any special knowledge of any variables that affect the heat transfer of the system should be considered in selecting the temperature measurement locations. Any known soil condition changes, heat sources like other pipelines, heat sinks like river crossings or anything else that could potentially be temperature significant heat source or heat sink should be measured. Then all of these measurements are weighted together based on the pipeline length that applied to each measurement to obtain an effective test medium temperature. Then the effective test medium is tracked all throughout the test as a means to explain the pressure variations that are observed during the test.

Now for an example of how the numbers work out. On a 10,000 ft, 8 inch pipeline with an effective test medium temperature of 60 °F, dP/dT = 17.38 PSI/°F. This means that for every 1 °F the weighted average test medium temperature rises, pressure should rise 17.38 PSI. If a recorded pressure rise of 10.0 PSI is observed throughout the test, is it a successful test? 17.38 PSI – 10.0 PSI = 7.38 PSI. What happened to this missing 7.38 PSI? One possible explanation is that there was a leak in the pipeline system. For this particular pipeline system, a 7.38 PSI loss in 8 hours represents a loss of 8 gallons. A loss of 8 gallons in 8 hours represents a loss of 209 barrels per year for this system; this is the potential leak size that could have been masked on this test without effective temperature change consideration.

Was the pipeline in the above example leaking, or was something else happening? Either the pipeline was leaking or one of the five basic assumptions of the leak test calculations was being violated so much that the quality of the data if not high enough to make an accurate leak test determination. The first explanation is that a leak is happening either above ground or below ground that is currently unseen. The second explanation is that for whatever reason, the temperature measurements the engineer has taken are not an accurate representation of the test medium, perhaps because the test medium did not have enough stabilization time to approach equilibrium with its surroundings. A third explanation is that perhaps there was excessive air in the line. Either way, the engineer cannot "pass" the test since there is a significant possibility of an unseen leak occurring.

When water from an outside source enters a pipeline system, it generally has to reach equilibrium with the earth temperature. Sometimes the temperature differential between the fill water and the earth temperature is over 30 °F. How long it takes for the water to reach equilibrium with its surroundings depends many factors including soil type, pipeline size, and temperature differential. The larger the pipe size, the longer it takes water to stabilize. The easiest way to tell if the water has stabilized is to take successive temperature measurements. Take some buried pipeline measurements before water is introduced in to the system. Then once water is introduced, take measurements until the temperature falls back to earth temperature, or nearly so.

Prior to the test medium reaching temperature equilibrium with the ground, there are large temperature gradients all throughout the pipeline system that are not observed with just a few buried temperature measurement locations. There will be large unexplained pressure changes during the test simply because the engineer does not have enough temperature data to accurately represent the pipeline test medium temperature.

By allowing the test medium to stabilize, the engineer can use fewer buried temperature measurements to approximate the entire buried pipeline temperature. In a relatively stable system, four to twelve buried pipeline temperature measurements can be an adequate representation of the entire several mile long pipeline system. Then, during the hydro-static test, small changes in buried pipeline temperature are accurately tracked and considered in the leak test evaluation calculations. Sometimes the stabilization period can be just overnight and sometimes it can take several days.

Comparison of Error in Pipeline Hydro-Static Test Measurement Systems

Additionally, in order to better conduct leak tests, a description of the error associated with the measurements commonly used during a test will be presented. By way of example, a comparison of the error associated two sets of measuring equipment will be presented:

- A hydro-static test conducted with a pressure *Barton* chart and a temperature *Barton* chart.
- A hydro-static test conducted with the digital instrument and automated data collection system as described above.

A sample *Barton* pressure chart has an accuracy of 1% of the full scale reading. The full scale range on a typical *Barton* chart is 0 – 3000 PSI. A sample *Barton* temperature chart has an accuracy of 1% of the full scale reading as well. The full scale range on a typical *Barton* chart is 0 – 150 °F.

There is some error in the measurement of pressure using a *Barton* chart and there is also some error in the measurement of temperature using a *Barton* chart. A full discussion of statistics is beyond the scope of this paper, but the overall uncertainty in the measurement system can be expressed in common terms by assuming the data set is a Gaussian distribution of data. (Bevington, 2013).

In order to make this comparison, some sample dP/dT values can be used to show the relative sizes of the error. **Table 1** is a representation of the uncertainty in the *Barton* chart measurement system showing the error in terms of temperature differential and pressure differential.

Table 1: *Barton* Chart Measurement System Overall Uncertainty

Pressure (PSI)	Pressure Error, +/- (PSI)	Temp. (°F)	Temp. Error, +/- (°F)	dP/dT (PSI/°F)	Overall System Error, +/- (PSI)	Overall System Error, +/- (°F)
2000.0	30.0	60.0	1.5	31.2	55.6	1.78
2000.0	30.0	100.0	1.5	39.8	66.8	1.68
440.0	30.0	60.0	1.5	31.2	55.6	1.78
440.0	30.0	100.0	1.5	39.8	66.8	1.68
1000.0	30.0	60.0	1.5	31.2	55.6	1.78
1000.0	30.0	100.0	1.5	39.8	66.8	1.68

Both of the ways of describing the error in this measurement system are equivalent. The result of using the *Barton* pressure chart and temperature chart measurement system is that a hydro-static test has an effective uncertainty of 55.6 – 66.8 PSI in these examples, or 1.68 – 1.78 °F. As can be seen in the above examples, a temperature or pressure differential of this size can mask a sizable potential leak size, potentially over 200 barrels per year in the example presented above.

A sample digital *Crystal* gauge has an accuracy of 0.1% of the pressure reading. The sample *Aultrol* temperature transmitters have an accuracy of 0.1% of the full scale. The full scale range on these devices is 0 – 150 °F. The sample RTDs have an accuracy of 0.3 °F + 0.5% of temperature reading.

Table 2 is a representation of the uncertainty in the digital instrument & automated data collection measurement system showing the error in terms of temperature differential and pressure differential, in the same manner as for the *Barton* chart measurement system.

Table 2: Digital Instrument & Automated Data Collection Measurement System Overall Uncertainty

Pressure (PSI)	Pressure Error, +/- (PSI)	Temp. (°F)	Autrol Error, +/- (°F)	RTD Error, +/- (°F)	dP/dT (PSI/°F)	Overall System Error, +/- (PSI)	Overall System Error, +/- (°F)
2000.0	2.0	60.0	0.15	0.6	31.2	19.4	0.62
2000.0	2.0	100.0	0.15	0.8	39.8	32.5	0.82
440.0	0.4	60.0	0.15	0.6	31.2	19.3	0.62
440.0	0.4	100.0	0.15	0.8	39.8	32.4	0.81
1000.0	1.0	60.0	0.15	0.6	31.2	19.3	0.62
1000.0	1.0	100.0	0.15	0.8	39.8	32.4	0.81

The result of using the digital instrument and automated data collection measurement system is that a hydro-static test has an effective uncertainty of 19.4 – 32.5 PSI in these cases, or 0.62 – 0.82°F.

As can be seen from **Tables 1 - 2**, the error is two to three times greater with the *Barton* chart measurement system than with the digital instrument & automated data collection system. Depending on the pipeline system, the difference in error between the uncertainties associated with these two measurement systems could mask a significant leak. This is one reason why the digital instrument & automated data collection system was developed, to remove some of the error in the measurement system and to make evaluating a leak test easier.

Conclusion

Using proper data collection methods gives the engineer a powerful tool in the process of leak test evaluation during a hydro-static test. If the engineer is able to control the five basic assumptions of the leak test calculations, then it is possible for even relatively inexperienced engineers to conduct and certify the successful completion of leak tests with repeatable results and a justifiable mathematical basis.

The price of failure is high in the evaluation of leak tests; there are significant consequences from putting a leaking pipeline into hydrocarbon service. Therefore, the engineer must be cautious in the evaluation of leak tests and in accepting that a hydro-static test has passed the leak test component of the test. Buried manufacturing pipeline defects have been found using these methods. Above ground leaking connections at flanges and valves beyond counting have also been identified using these methods. All of these leaks would go un-noticed without a careful leak test giving consideration to the above ground and below ground system temperature at multiple measurement locations in the pipeline system.

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Maximum Transient Pressures in Batch Pipelines due to Valve Closures

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Abstract

In a batched pipeline, the transient pressure due to a rapid valve closure depends on batch sequences, batch interface locations, and batch volumes. Since an infinite number of potential batch scenarios exist, it is necessary to identify a worst-batch scenario where the maximum transient pressure occurs for the transient event of the valve closure. This study demonstrates that the worst-batch scenario is the densest fluid filling the pipeline between the closed valve and its upstream boundary and the least dense fluid filling the pipeline between the closed valve and its downstream boundary. For a surge-dominated transient event, the worst-batch scenario could be a smallest batch of the densest fluid placed just upstream of the closed valve with the least dense fluid filling the remaining volume of the pipeline.

INTRODUCTION

Different types of liquid petroleum products are transported within the same pipelines in batches. As the different fluids enter a pipeline, the batches move continuously along the pipeline. When a valve closes, the transient pressure caused by the valve closure depends on the batch sequences; the batch interface locations; and the batch volumes. Since an infinite number of potential batch scenarios exist, it is necessary to identify a worst-batch scenario where the maximum transient pressure occurs for the valve closure. This study will present the "worst" batch scenario where the maximum transient pressure occurs due to the valve closure.

SURGE AND LINE PACK

In the event of a rapid flow stoppage due to a valve closure, surge pressure is represented by the basic water-hammer equation:

$$\Delta p = \rho a V \tag{1}$$

Δp - surge pressure
ρ - density of the fluid

a - wave speed
V - initial fluid velocity.

Ignoring the effect of the pipe-wall elasticity, the wave speed is $\sqrt{K/\rho}$ where K is the bulk modulus of elasticity of the fluid and Equation 1 becomes:

$$\Delta p = \sqrt{K}\sqrt{\rho}V \tag{2}$$

The surge pressure is proportional to the bulk modulus of fluid, the density of fluid, and the flow velocity. The flow velocity, dependent on the pipeline system and its operations, can be derived from Darcy's equation:

$$V = \sqrt{\frac{2DgH}{fL}} \tag{3}$$

D - pipe diameter
H - energy head provided by the pipeline system
f - friction factor
L - pipe length.

The friction factor depends on the Reynolds number of $Re = DV/\upsilon$ and the relative pipe roughness of $\frac{2\epsilon}{D}$ where υ is the kinematic viscosity of the fluid and ϵ is the specific roughness of the pipe. Typically, the higher the fluid density, the higher the bulk of modulus and viscosity are. The higher the viscosity, the lower the flow velocity is.

When two different fluids (A and B) are transported separately in the same pipeline with the same energy head of H, the surge pressure ratio of $\Delta p_A/\Delta p_B$ can be represented as:

$$\frac{\Delta p_A}{\Delta p_B} = \sqrt{\frac{K_A}{K_B}}\sqrt{\frac{\rho_A}{\rho_B}}\frac{V_A}{V_B} = \sqrt{\frac{K_A}{K_B}}\sqrt{\frac{\rho_A}{\rho_B}}\sqrt{\frac{f_B}{f_A}} \tag{4}$$

In fully turbulent flow (very high Reynolds number), the friction factor depends only on the relative roughness of $1/\sqrt{f} = 1.74 - 2\,log_{10}\left(\frac{2\epsilon}{D}\right)$ (the Karman-Nikuradse equation). The surge pressure ratio becomes:

$$\frac{\Delta p_A}{\Delta p_B} = \sqrt{\frac{K_A}{K_B}}\sqrt{\frac{\rho_A}{\rho_B}} \qquad \text{for} \qquad \frac{V_A}{V_B} = \sqrt{\frac{f_B}{f_A}} = 1 \tag{5}$$

The above equation sets the maximum surge ratio of Fluid A vs. Fluid B for all batch scenarios.

For the transient event of a valve closure, Equation 2 explicitly indicates that the maximum surge pressure occurs in the worst-batch scenario where a smallest batch of the densest fluid placed just upstream of the closed valve (equivalent to the largest fluid density and bulk of modulus) and the least dense fluid filling the remaining volume of the pipeline (equivalent to the highest initial flow velocity). Since the transient pressure is composed of the steady-state pressure, the surge pressure, and the line pack pressure, the worst-batch scenario specified above is applicable only if the transient event is surge-dominated (or line pack is negligible). A surge-dominated event usually occurs in short pipelines with low friction where a sudden valve closure could cause its upstream flow motion to stop.

In long pipelines, the sudden valve closure could not completely stop the flow motion between the closed valve and its upstream boundary. An additional pressure increase occurs over the surge pressure due to line pack, which is related to the flow momentum upstream of the closed valve; the attenuation of the pressure wave-front; and the hydraulic impedance at the batch interfaces. Clearly, the valve's location plays an important role in line pack. When the valve is close to its upstream boundary, the line pack is less. When the valve is far away from its upstream boundary, line pack can be significant. The following study demonstrates that the worst-batch scenario is the densest fluid filling the pipeline between the closed valve and its upstream boundary and the least dense fluid filling the pipeline between the closed valve and its downstream boundary if line pack is significant in the transient event.

QUASI-STEADY STATE STUDY

A hydraulic model of a hypothetical pipeline was developed to investigate the pressure transients caused by the valve closures using the Synergi Pipeline Simulator (SPS – formally known as Stoner Pipeline Simulator). The hydraulic model includes a horizontal pipeline, upstream and downstream boundaries, and five motor-operated valves (MOVs). The pipeline length is 40 km with a nominal diameter of 400 mm. The upstream and downstream boundaries have constant pressures of 2500 and 50 kPag, respectively. Five valves B1, B2, B3, B4, and B5 are located at a distance of 0, 10, 20, 30, and 40 km from the upstream boundary. Diffusion at the batch interfaces is ignored for simplification.

This study uses diesel and gasoline as an example. The typical properties of diesel and gasoline at atmospheric pressure and 15.6^{o}C are presented in Table 1.

Table 1. Typical Fluid Properties of Diesel and Gasoline

Product	Specific Gravity	Viscosity (cSt)	Bulk Modulus (MPa)
Diesel	0.85	2.68	1172
Gasoline	0.74	0.48	862

In this study, it is assumed that the pipeline transports diesel and gasoline in batches.

- When a diesel batch follows a gasoline batch, it is defined as diesel/gasoline sequence.

- When a gasoline batch follows a diesel batch, it is defined as gasoline/diesel sequence.

This study considers a two-batch operation (only two batches in the pipeline) and the batch scenarios are defined as:

- Die25/Gas75 batch scenario
  - a diesel/gasoline sequence,
  - diesel and gasoline batch volumes as 25% and 75% of the total pipeline's volume,
  - and diesel/gasoline interface located at a distance of 10 km from the upstream boundary.
- Gas25/Die75 batch scenario
  - a gasoline/diesel sequence,
  - gasoline and diesel batch volumes as 25% and 75% of the total pipeline's volume,
  - and gasoline/diesel interface located at a distance of 10 km from the upstream boundary.

This study also considers a three-batch operation and the batch scenario is defined as:

- Gas25/Die25/Gas50 batch scenario
  - a gasoline/diesel/gasoline sequence,
  - gasoline, diesel, and batch volumes as 25%, 25% and 50% of the total pipeline's volume,
  - and gasoline/diesel interface and diesel/gasoline interface located at a distance of 10 km and 20 km from the upstream boundary.

For the diesel/gasoline sequence, five quasi steady-state cases were developed for five batch scenarios (Die0/Gas100, Die25/Gas75, Die50/Gas50, Die75/Gas25, and Die100/Gas0), one case for one batch scenario. For the gasoline/diesel sequence, five quasi steady-state cases were developed for five batch scenarios (Gas100/Die0, Gas75/Die25, Gas50/Die50, Gas75/Die25, and gasoline Gas100/Die0).

Table 2 presents the initial flow velocities for the two-batch operation with all ten batch scenarios. It can be seen that the initial flow velocity becomes smaller when diesel batch volume increases or gasoline batch volume decreases in the pipeline.

Table 2. Quasi Steady-State Flow Velocities (m/s)

Die0/Gas100 Gas100/Die0	Die25/Gas75 Gas75/Die25	Die50/Gas50 Gas50/Die50	Die75/Gas25 Gas25/Die75	Die100/Gas0 Gas0/Die100
2.21	2.12	2.04	1.96	1.90

For gasoline/diesel/gasoline sequence, four quasi steady-state cases were developed for four batch scenarios:

- Gas0/Die50/Gas50
- Gas25/Die25/Gas50,

- Gas35/Die15/Gas50,
- Gas45/Die5/Gas50.

TRANSIENT ANALYSIS

All transient events are initialized from a quasi-steady state case by closing a valve instantaneously at 0.1 minute.

For the diesel/gasoline sequence, a total of 25 transient events were performed for 5 batch scenarios in a parametric study. For each transient event, one valve closes starting from a quasi-steady state case. Table 3 presents the highest transient pressures that occur on the upstream side of the closed valve for these 25 transient events. Five batch scenarios are defined in the header row of Table 3. For an example, the transient event, defined by B3 closure and batch scenario Die50/Gas50, has a highest transient pressure of 4196 kPag. The maximum transient pressure for B3 closure is 4196 kPag for all five batch scenarios as highlighted in green. The results show that:

- The maximum pressure always occurs when the diesel/gasoline interface is at the valve which was closed.
- The maximum pressure always occurs when diesel fills the pipeline between the closed valve and its upstream boundary and gasoline fills the pipeline between the closed valve and its downstream boundary.

Table 3. Highest Transient Pressures (kPag) – Diesel/Gasoline Sequence

Valve	Die0/Gas100	Die25/Gas75	Die50/Gas50	Die75/Gas25	Die100/Gas0
B1	4513	4432	4357	4289	4227
B2	4111	4369	4289	4239	4179
B3	4022	4124	4196	4139	4090
B4	3881	3899	3907	3992	3953
B5	3693	3651	3609	3667	3770

For the gasoline/diesel sequence, Table 4 presents similar highest pressures that occur on the upstream side of the closed valve for a total of 25 transient events. It can be seen that:

- The maximum pressure (in yellow) for the gasoline/diesel sequence is always lower than that (in green, Table 3) for the diesel/gasoline sequence for the same valve closure. Because of that, the gasoline/diesel sequence will not be studied further.
- The maximum pressure does not occur when the gasoline/diesel interface is at the valve that was closed.

Table 4. Highest Transient Pressures (kPag) – Gasoline/Diesel Sequence

Valve	Gas0/Die100	Gas25/Die75	Gas50/Die50	Gas75/Die25	Gas100/Die0
B1	3924	3975	4031	4092	4161
B2	4179	3938	3987	4043	4113
B3	4092	3925	3917	3969	4024
B4	3954	3865	3876	3841	3884
B5	3770	3760	3762	3759	3695

For the gasoline/diesel/gasoline sequence, Table 5 presents the highest pressures that occur on the upstream side of valve B3 for four batch scenarios. The maximum pressure (in green) is 4196 kPag among all simulated batch scenarios and it occurs when diesel fills the pipeline between the closed valve and the upstream boundary.

Table 5. Highest Transient Pressures (kPag) – Gasoline/Diesel/Gasoline Sequence

Batch Scenarios	Highest Transient Pressure
Gas0/Die50/Gas50	4196
Gas25/Die25/Gas50	4011
Gas35/Die15/Gas50	4030
Gas45/Die5/Gas50	4036

RESULT ANALYSIS – B3 CLOSURE

The valve B3 closure is used as an example to analyze the transient pressures in details. Figure 1 presents the transient pressure on the upstream side of valve B3 for the batch scenario Die50/Gas50. The pressure surges rapidly to about 2900 kPag at 0.1 minute when valve B3 closes. After the initial pressure spike, the pressure continues to climb to its highest pressure of about 4200 kPag due to line pack. The simulated results are:

- initial steady-state pressure about 1100 kPag,
- surge pressure about 1800 kPag,
- line pack pressure about 1300 kPag,
- highest transient pressure about 4200 kPag.

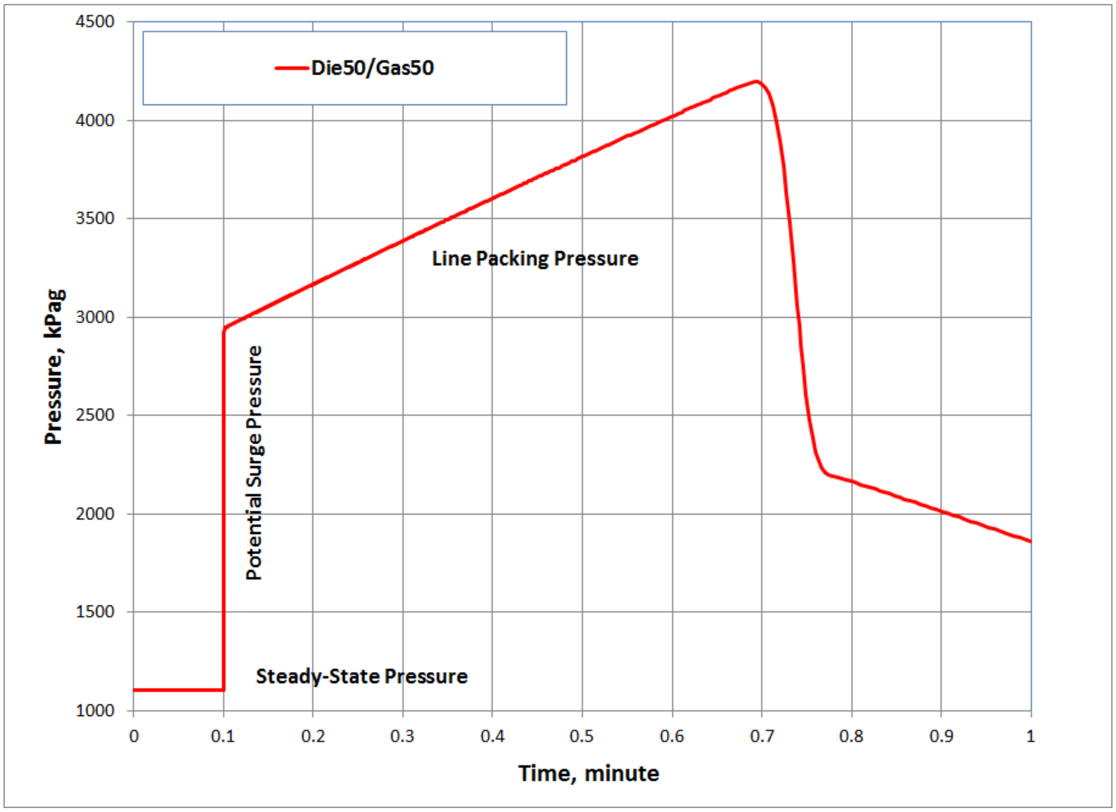

FIGURE 1. B3 Inlet Transient Pressures - Diesel/Gasoline Sequence

For a diesel/gasoline sequence and valve B3 closure, Table 6 presents the highest transient pressures and their three components on the upstream side of B3 for 5 batch scenarios. Figure 2 presents the transient pressures for three batch scenarios. The simulated results are:

- The maximum transient pressure (in green) occurs at the worst-batch scenario of Die50/Gas50 with the diesel/gasoline interface located at B3 and diesel filling the pipeline between the closed valve B3 and its upstream boundary.
- The worst-batch scenario Die50/Gas50 has the lowest steady-state pressure among all 5 batch scenarios because of hydraulic grade-lines.
- The worst-batch scenario Die50/Gas50 has the largest surge pressure among all 5 batch scenarios, which could be explained by referring to Equation 2:
    - For these three batch scenarios of Die50/Gas50, Die75/Gas25, and Die100/Gas0, all of these three batch scenarios have the same density and bulk of modulus but the worst-batch scenario of Die50/Gas50 has the highest initial flow velocity (see Table 2).
    - For these three batch scenarios of Die0/Gas100, Die25/Gas75, and Die50/Gas50, the worst batch scenario of Die50/Gas50 has the higher density and bulk of modulus.

- The line pack pressure for the worst-batch scenario Die50/Gas50 is the second highest among all 5 batch scenarios.
    - Line pack depends on the fluid momentum between the closed valve and its upstream boundary and the pressure wave attenuation, which determines the pressure increase slope.
    - Line pack also is related to the hydraulic impedance at the diesel/gasoline interfaces which results in the pressure jump for the batch scenario of Die25/Gas75.

Table 6. Highest Pressure and Three Components (kPag) – Valve B3 Closure and Diesel/Gasoline Sequence

Pressure (kPag)	Die0/Gas100	Die25/Gas75	Die50/Gas50	Die75/Gas25	Die100/Gas0
Highest Pressure	4022	4124	4196	4139	4090
Steady-state	1276	1183	1102	1194	1276
Surge	1626	1559	1822	1758	1697
Line pack	1120	1382	1272	1187	1121

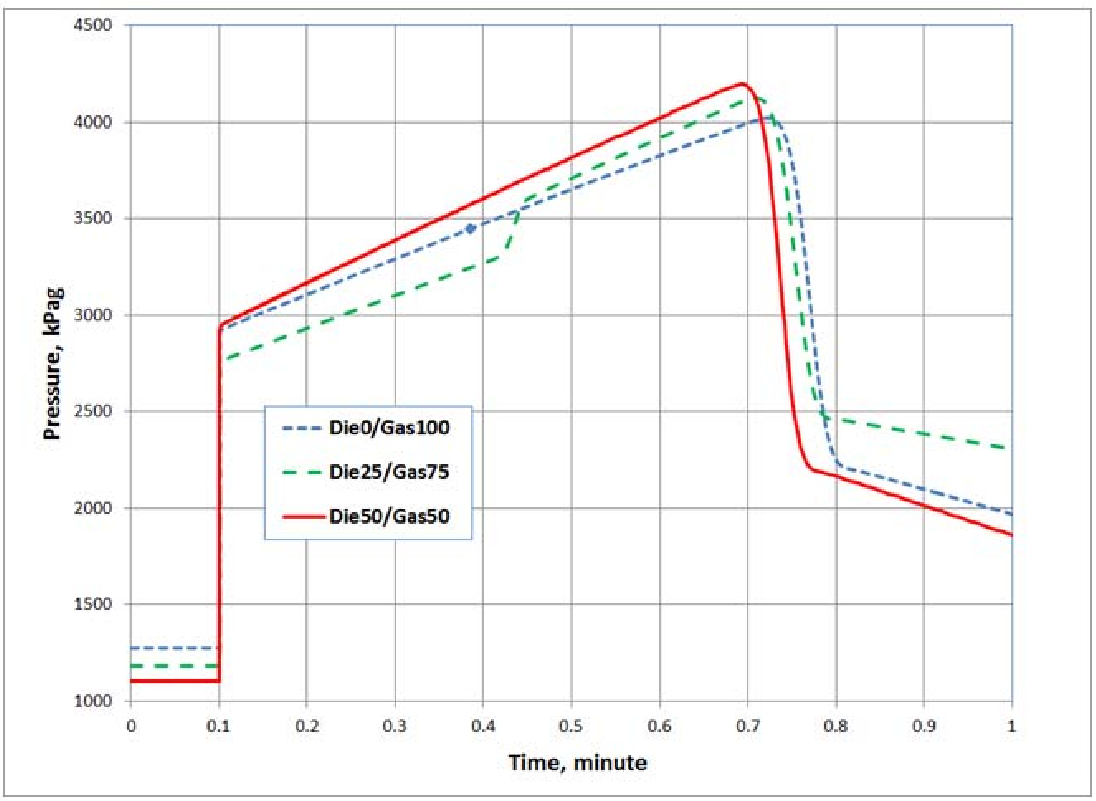

FIGURE 2. B3 Inlet Transient Pressures – Diesel/Gasoline Sequence

RESULT ANALYSIS – B3 CLOSURE

Table 7 presents the highest transient pressures and the three pressure components for a B3 closure for a gasoline/diesel/gasoline sequence. Figure 3 presents the transient pressures at B3 for a gasoline/diesel/gasoline sequence. The simulated results are:

- The maximum transient pressure (in green) occurs at the worst-batch scenario of Gas0/Die50/Gas50 with the diesel/gasoline interface located at B3 and diesel filling the pipeline between the closed valve B3 and its upstream boundary.
- The worst-batch scenario Gas0/Die50/Gas50 has the lowest steady-state pressure among all 4 batch scenarios because of hydraulic grade-lines.
- The worst-batch scenario Gas0/Die50/Gas50 has the smallest surge pressure among all 4 batch scenarios since it has the lowest initial flow velocity and the same density and bulk of modulus in comparison with other three batch scenarios.
- The worst-batch scenario Gas0/Die50/Gas50 has the largest line pack pressure among all 4 batch scenarios:
    - The worst-batch scenario has the highest fluid momentum between the closed valve and its upstream boundary, resulting in the steeper pressure increase in comparison with other three batch scenarios.
    - The hydraulic impedance has no effects on the line pack pressure in the worst-batch scenario since there is no interface change in the pipeline between the closed valve and its upstream boundary. The hydraulic impedance results in the line pack pressure drop when the gasoline/diesel interface reaches the closed valve for all other three batch scenarios.

Table 7. Highest Pressure and Three Components (kPag) – Valve B3 Closure & Gasoline/Diesel/Gasoline Sequence

Batch Scenarios	Highest Pressure	Steady-state Pressure	Surge Pressure	Line pack Pressure
Gas0/Die50/Gas50	4196	1102	1822	1272
Gas25/Die25/Gas50	4011	1184	1904	923
Gas35/Die15/Gas50	4030	1220	1929	881
Gas45/Die5/Gas50	4036	1258	1963	815

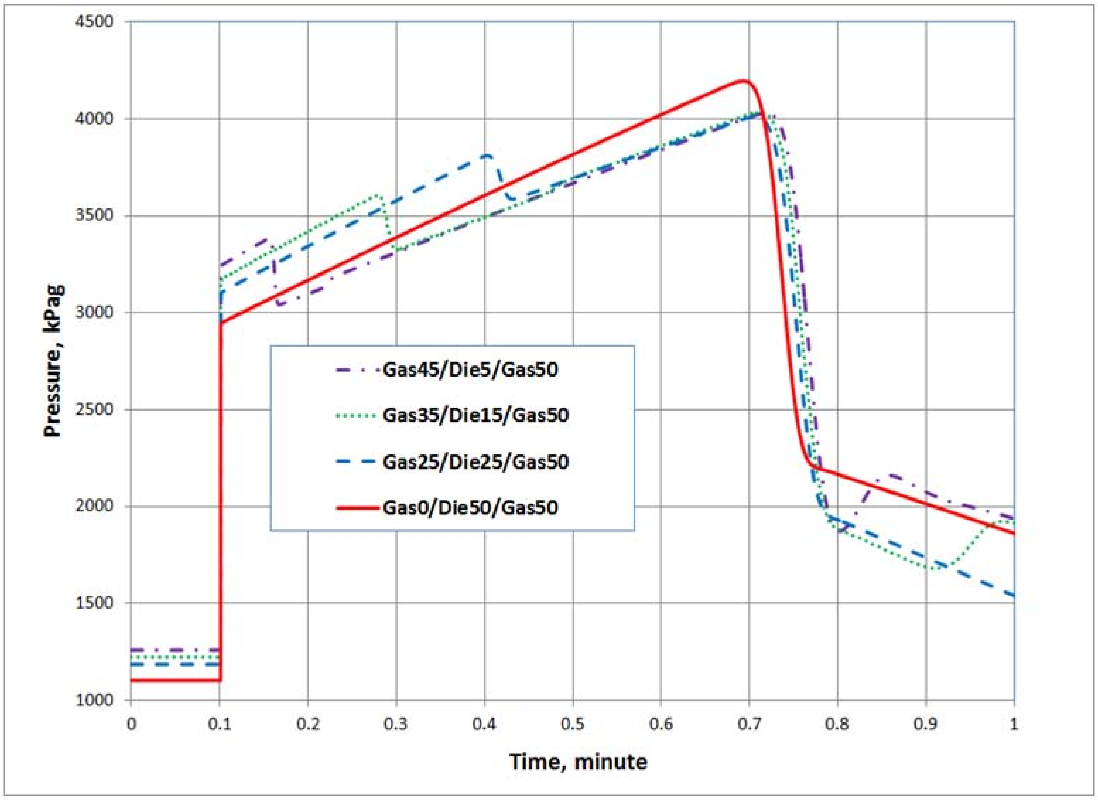

FIGURE 3. B3 Inlet Transient Pressures - Gasoline/Diesel/Gasoline Sequence

DISCUSSIONS AND CONCLUSIONS

The transient pressure consists of the steady-state pressure, the surge pressure, and the line pack pressure. The magnitude of the transient pressure is dependent on the batch sequences, the batch interface locations, and the batch volumes. Since an infinite number of potential batch scenarios exist, it is necessary to identify a worst-batch scenario that results in the maximum transient pressure for a valve closure.

Typically, the worst-batch scenario has the most dense fluid filling the pipeline between the closed valve and its upstream boundary and the least dense fluid filling the pipeline between the closed valve and its downstream boundary. For a surge-dominated transient event, the worst-batch scenario is a smallest batch of the most dense fluid placed just upstream of the closed valve and the least dense fluid filling the remaining volume of the pipeline.

To determine whether a transient event is a surge-dominated transient event or not, we need to consider both the pipeline characteristic and the valve location. The surge pressure usually dominates for short pipelines with low friction and/or for long pipelines where the valve is very close to its upstream boundary.

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Electrochemical Impedance Spectroscopy: Characterizing the Performance of Corrosion Protective Pipeline Coatings

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Abstract

Electrochemical Impedance Spectroscopy, EIS, measures how the frequency dependent electrical impedance of a coating film changes during its exposure to corrosive media. The technique is fairly rapid, often non-destructive and is sensitive to changes in the coating and can also be performed in the field. The electrical properties of the paint film are measured through its thickness between the metal and an exterior electrode which is usually a reservoir of a conductive salt solution on the outer surface. Polymer pipeline coatings and linings are designed to be barriers to water and dissolved salts that would otherwise corrode the steel water pipes. High values of impedance are characteristic of coatings without defects and that do not absorb water or salts. Diminished values of impedance show that either a defect has formed or that the coating has absorbed electrolyte that may eventually corrode the metal pipe substrate. The technique was used to compare the performance of modern epoxy or polyurethane coatings with coat tar enamel that is known to have a much extended service life, if undisturbed. Results show that, after one and a half years, all the coatings are performing well although the polyurethane coatings are performing better than the epoxies.

INTRODUCTION

Many coatings are employed to protect a substrate from aggressive chemicals that may corrode that substrate. The most common type of substrate that requires protection in this way is usually steel or aluminum alloy. The aggressive chemicals that are most often of concern are water and ionic salts from the environment, e. g. sodium chloride from ocean spray or salts from ground water. Often coatings are used in combinations where the primer, next to the substrate, contains corrosion inhibitors. Typically, there is a topcoat over this primer that not only provides the required appearance but also is intended to be the barrier to water etc. If water eventually gets through the topcoat, then the corrosion inhibitors in the primer can limit or arrest the corrosion of the metal substrate. Here, only the barrier properties are discussed because steel water pipe is protected by a barrier coating, there is normally no corrosion inhibiting primer.

Modern, health and environmental conscious use of coatings has lead to the use of epoxy or polyurethane coatings than can easily be applied, carefully, in the field or factory without the need for specialized health or environmental protection. However, it is well recognized that coal tar enamel coatings, if undisturbed, provide very long term protection that is difficult for modern coatings to achieve. Several liquid epoxy and polyurethane coatings were investigated here, as well some samples of coal tar enamel coating.

ELECTROCHEMICAL IMPEDANCE SPECTROSCOPY

Background

Water and aggressive salts are ionic, and therefore very polar, and thus can be detected by the change they bring to the electrical properties of an organic coating which would otherwise be a good electrical insulator. Good insulators have very high values of electrical resistance because they do not permit much current to flow and they are good dielectrics for making capacitors because they store electrical energy by preventing the charged species from moving and thus being neutralized by meeting the opposite charge. Measuring the electrical resistance and capacitance of a barrier coating is thus a direct method of assessing whether it is permitting charged species to flow, and a measure of its protective properties. If a voltage is applied to a coating film and the resultant current is measured as a function of the frequency of the applied voltage, then we obtain a spectrum of the current with frequency. Instead of using the simple Ohm's law to identify a resistance that is independent of the applied frequency, this spectrum measures an 'impedance' where the variation with frequency can be used to identify, separately, the resistive and capacitative natures of the coating.

If the measurement of electrical properties is done without water or salts, then the measurement is 'dielectric spectroscopy' of the intrinsic electrical properties of the material. If the electrical properties are measured when water and ionic material can make their way through the coating and cause corrosion of the metal underneath, i.e. cause electrochemical changes to the metal, and thus to the measurement, then the experiment is termed 'electrochemical impedance spectroscopy'.

Electrical Properties

If a resistor is made of a material with a characteristic electrical resistivity of ρ ohm.meter (Ω.m) and has a cross section area of A m^2, and path length for the current of l m, then its overall resistance, R ohm(Ω) , is:

$$R = \frac{\rho l}{A} \qquad 1$$

If a coating is thought of as a resistor, then a thicker (greater l) coating has a greater resistance to the passage of charge. Polymers tend to have a very high resistivity, between 2 x 10^{11} and 2 x 10^{13} Ω.m. Resistivity is independent of the frequency and the current through a resistor rises and falls exactly in step with the voltage.

The capacitance, C, of the same coating can be described in a similar way. In this case, the material property is its relative permittivity (formerly, and often, called "dielectric constant"), ε, which depends on the frequency of the applied voltage, although for polymers there is usually little change until frequencies of $>10^5$ Hz.

$$C = \frac{\varepsilon \varepsilon_0 A}{l} \qquad 2$$

ε_0 is the permittivity of a vacuum, 8.854 x 10^{-12} (F/m, farads per meter or Coulombs per volt per meter, C/V/m). Polymers have a relative permittivity of ~ 3 – 10 (dimensionless). The current through a capacitor lags behind the applied voltage and for a sinusoidal voltage, the phase lag is a quarter of a whole oscillation (90° or π/2).

Fortunately, aqueous electrolytes that may corrode metals have very low resistivity and very high relative permittivity, so they are easy to detect while measuring the electrical properties of a polymer coating; resistance drops as the electrolyte permeates the coating and the capacitance increases. EIS can detect changes to a coating in a corrosive environment that are not readily apparent by other means.

EIS Measurement on coatings

In order to measure the electrical properties of a coating, the metallic substrate is employed as one electrode attached to the coating underneath, and the electrode on the other (top) surface is made of a cylinder containing (usually) the conductive aqueous test electrolyte. The electrolyte is conductive, so the electronic device that applies the voltage and measures the current is connected by merely inserting a wire into the electrolyte. The impedance (voltage ÷ current) is measured over a range of frequencies, from 0.01 Hz to 10^5 Hz. Note that the original name for this technique was AC (alternating current) impedance spectroscopy.

A corroding metal has a characteristic potential (~ 1 V depending on the metal and circumstances) generated from the difference in charge between the metal and the charged ions that have dissolved into the electrolyte (open circuit potential, i.e. measured under zero current conditions). Since the absolute potential of a single metal electrode cannot be measured in isolation, a reference electrode is used in the circuit so that this characteristic potential can be measured and so that alternating voltage (5 -10 mV) is imposed only as a small perturbation above and below this characteristic potential. Thus the experiment harmlessly measures the electrical properties of the materials but does not change the electrochemical processes. If the experiment is stopped and the electrolyte removed from the coating surface, the coating should be unaffected by the experiment, unless the electrolyte permanently affects the coating. In principle, EIS is a non-destructive test. A schematic diagram is shown in figure 1.

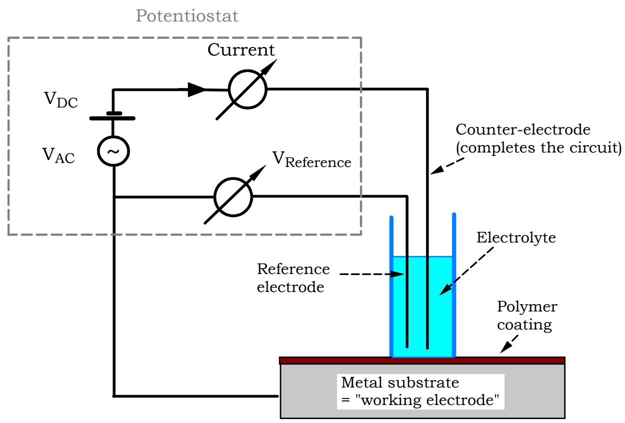


Figure 1. EIS experiment. The electrolyte is contained in a non-metallic cylinder that is sealed to the surface of the coating under investigation.

The 'potentiostat' maintains the potential of the working electrode constant with respect to the reference electrode by adjusting (and thereby measuring) the current at the counter electrode. The electrolyte must be conductive, but it can be chosen as either a standard solution, e.g. sodium chloride, or some other solution that is representative of the coating's exposure conditions. The counter-electrode completes the circuit to the testing equipment and is often made of a material that does not corrode, e.g. platinum or graphite. Modern potentiostats are small and work in conjunction with laptop computers and can be taken into the field and used if suitable connections can be made. In fact there is a variant that does not need a connection to the metal substrate but uses another electrolyte-like connection to a neighboring area of the coating.

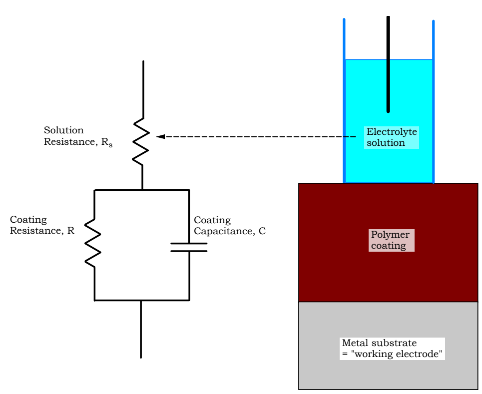


Figure 2. Simplest equivalent circuit for a coating that shows its resistive component, R, and its capacitative component, C.

The equivalent circuit, for a simple coating over a metal that is not corroding, is a "Randle's" circuit, figure 2. The coating resistance often has the subscript 'p', because, depending on its supposed origin, it is also known as the 'polarization' or 'pore' resistance.

If the experiment is continued until the metal starts to corrode, then the experimental results show new features and more components are included in the equivalent circuit to represent the corrosion processes and a corrosion layer on the metal surface. There are a variety of ways to represent this in an equivalent circuit, with the most common being another parallel resistance and capacitance combination in the coating resistance arm of the coating circuit above. This paper focuses on the properties of the coating, before corrosion, so no discussion of that is included here.

The solution resistance, R_s, is the resistance of the electrolyte solution between the counter electrode and the coating surface and is usually extremely small compared to the impedance of the coating and can be eliminated from the experimental data. The impedance, Z, of the simple equivalent circuit has a component that is in-phase with the excitation (voltage) and a component that is out of phase:

$$Z_{In-phase} - R_S = \frac{R}{1+(\omega CR)^2} \qquad \text{3a}$$

Solution resistance is included here for completeness, but ignored hereafter.

$$Z_{Out-phase} = \frac{-\omega CR^2}{1+(\omega CR)^2} \qquad \text{3b}$$

If the AC frequency of the test voltage is f, then $\omega = 2\pi f$. 'Modulus' is the absolute size of a quantity, regardless of its sign. If we ignore the solution resistance, the modulus of the impedance, $|Z|$, is given by the square root of the sum of the squares of the two components of the impedance:

$$|Z| = \frac{R}{\left[1+(\omega CR)^2\right]^{1/2}} \qquad \text{4}$$

At very low frequencies, $\omega \to 0$ and $|Z|$ becomes just the coating resistance, R. At high frequencies, the impedance becomes that of the equivalent capacitance only. Thus another advantage of the technique is that it can identify the two parts of the behavior. At low frequencies, the phase lag between current and voltage is very low, like a resistor, and at high frequencies it becomes 90 degrees, like a capacitor. For a resistor and capacitance combined in parallel, the phase lag is given by:

$$Phase\ lag = \theta = \tan^{-1}(\omega CR) \qquad \text{5}$$

If there are stratified coatings or a layer of corrosion under the coating, then these equations include more terms related to more equivalent circuit elements, and become very complicated and more difficult to understand. However, the simple description above is very useful for discussion.

In terms of the electrical properties of the coating material, equation 4 becomes:

$$|Z| = \frac{l}{A} \cdot \frac{\rho}{\left[1+(\omega\varepsilon\varepsilon_0\rho)^2\right]^{1/2}} = |\mathrm{E}|_{Material} \cdot \frac{l}{A} \qquad 6$$

If $|Z|$, the impedance (ohms), is the quantity presented, its value depends on the area of the test cell (A) and on the coating thickness (l). Thicker coatings give higher values of impedance which expresses that they should be better barriers. However, if one can measure the thickness of the coating and the area of the test cell, then one can deduce $|E|_{Material}$ which represents the dielectric properties of the coating material. $|E|_{Material}$, has units of ohm.meter and has in-phase and out-of-phase components itself that are easily deduced from equations 3a and 3b.

There are two graphical ways to view the behavior for analysis. Most often a 'Bode" plot of $|Z|$ as a function of voltage frequency is plotted on a logarithmic graph, see below. It is also common to plot the phase lag (angle), θ, as a function of frequency on the Bode plot. One can see the transition from resistive behavior to capacitative behavior from both impedance and phase angle data.

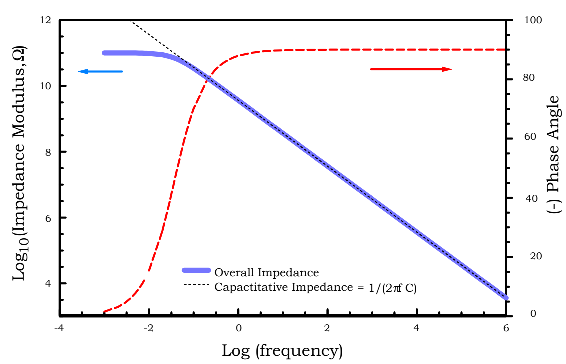


Figure 3. Bode plot with resistance and capacitance representative of a thick, purely polymer coating with coating resistance 10^{11} Ω and capacitance 0.04 nF. Note, the current lags voltage in a capacitor, so the phase angle is negative.

Another common approach is the 'Nyquist' plot where the imaginary part of the impedance is plotted as a function of the real part (the abscissa). This has some uses although it obscures the frequency of measurement. One can show after some algebra in which ω is eliminated between equations 3a and 3b that for the simple parallel RC circuit that the relationship is semi-circular, with a radius of $R/2$ and a center at (R/2, 0). One can also include the solution resistance in this plot which shifts the real component values to higher values by its magnitude.

Effect of Water on a Coating, as Measured by EIS

One of the principal applications of EIS in coatings is to determine whether there is significant water imbibed that can eventually threaten corrosion of the steel water pipe. There have been detailed discussions of the effects of water on the resistive and capacitative properties of coatings [Stafford 2006, Hinderliter 2006], but only a common, simple approach is outlined here. The Brasher-Kingsbury model [Brasher 1954] assumes that a random and even distribution of water in a coating changes the relative permittivity according to:

$$\varepsilon_{New} = \varepsilon_{Polymer} . \varepsilon_{Water}^{\phi} \qquad 7$$

Where ϕ is the volume fraction of water that has made its way into the polymer. Thus the capacitance is increased via the increased relative permittivity:

$$C_{New} = C.\left(\varepsilon_{Water}^{\phi}\right) \qquad 8$$

The relative permittivity of water is usually taken to be 80 at room temperature, which is very different from that of most polymers. For example, the coating capacitance would be increased by almost 25% if the polymer had taken on 5% by volume of water. Often the amount of water that a coating has absorbed is calculated from EIS results where the capacitance of the coating before and after immersion is measured and equation 8 is inverted to deduce the value of ϕ. There have been many attempts to find a more exact expression for the amount of water uptake [Moreno 2012] but the Brasher-Kingsbury equation is most often used because it is simple and greater accuracy is seldom required. It is more difficult to estimate the effect of water on resistivity [Stafford 2006, Hinderliter 2006] and seldom attempted.

RESULTS AND DISCUSSION

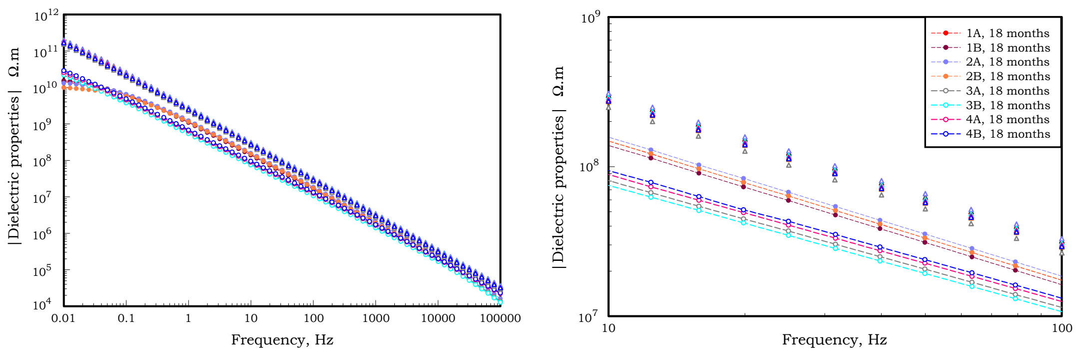
Coated steel panels were supplied by Northwest Pipe. One epoxy was 100% solids, i.e. solvent free, and the other was an 80% solids formulation. The epoxy and polyurethane coatings were applied both by factory automatic spray equipment and by hand held equipment characteristic of field-applied coatings. Coal tar enamel coating was applied by dipping the panels in the liquid coating. Coatings types and their average thickness (measured at Northwest Pipe) are given in table 1.

EIS was done by using an aqueous electrolyte of 3% sodium chloride in a cell of 5.31 cm^2 area. The data were taken with a Gamry Reference 600™ potentiostat connected to a standard desktop computer. The electrolyte was maintained in the cell, so the coatings did not dry out, but have been immersed continuously for 18 months approximately. Coatings used on steel water pipe are approximately 10 times thicker than industrial coatings that are often investigated in this way. This means that their impedance is an order of magnitude greater and so the current that must be detected is an order of magnitude less. There is some spurious scatter in the data at low frequencies (extremely high impedance) where inconsistencies in the electrical contact with the substrate and low frequency interference occurred.

Table 1. Pipeline coating samples.

Sample	Coating Type	Solids	Factory Equipment	Manual	Average Dry Film Thickness, mils
1A	Epoxy	100%		X	28.9
1B	Epoxy	100%		X	27.6
2A	Epoxy	100%	X		23.2
2B	Epoxy	100%	X		25.2
3A	Epoxy	80%		X	16.6
3B	Epoxy	80%		X	15.7
4A	Epoxy	80%	X		17.3
4B	Epoxy	80%	X		17.1
5A	Polyurethane	100%		X	23.5
5B	Polyurethane	100%		X	21.8
6A	Polyurethane	100%	X		30.8
6B	Polyurethane	100%	X		18.3
7A	Coal tar enamel	100%		*X	21.4
7B	Coal tar enamel	100%		*X	21.7
8A	Coal tar enamel	100%		*X	21.6
8B	Coal tar enamel	100%		*X	21.8

The Bode plots for the samples as the immersion period increased are shown below. In all cases, the initial values and the values at 18 months are plotted where the impedance has been multiplied by the area of the cell and divided by the thickness of the coating, i.e. reduced to (the modulus of) the dielectric properties, $|E|_{Material}$. There were many spectra taken at various periods but there was no further systematic change (that could be discriminated on these graphs) seen after 4 weeks immersion which is the same as the final results at 18 months. No evidence was seen of another process starting at any time, so it is reasonable to assume that no corrosion has occurred under any of these coatings after 18 months. There are 16 sets of data, so for clarity the data are presented in two groups. The first group is the epoxy samples and the second group is the polyurethanes and the coal tar enamel samples.



(a) (b)

Figure 4. EIS impedance of the epoxy coatings; (a) the whole frequency spectrum, (b) shows an expanded view of a middle portion of the same data. There is only one legend since the colors used are the same in both diagrams. In both diagrams the initial EIS spectra are depicted with unconnected triangle symbols and the results after 18 months immersion are depicted with circle symbols that are connected with a dashed line.

Figure 4a shows data that is completely typical of epoxy coatings. There is evidence of a flat plateau at low frequencies that indicates some resistive nature. The impedance clearly diminishes slightly at all frequencies after the immersion exposure due to water uptake within the coatings. However, all these coatings show very good impedance characteristics and no evidence of corrosion of the steel substrate. The expanded view, figure 4b, shows also that, in this particular group of coatings, the 100% epoxy had slightly higher impedance (dielectric) properties than did the 80% solids type. Thus potentially, one can regard this 100% epoxy coating as a slightly superior barrier material than this 80% formulation. This may not always be true, since the actual performance of any class of polymer coating depends on the other ingredients in the coating as well as the quality and thickness of the applied coating. In fact, for the 80% solids epoxy dry coating, its thicknesses were systematically thinner in this study and so would provide less of an ultimate barrier (assessed in this way by EIS) than the thicker coatings made using the 100% solids epoxies. There is no trend according to whether the epoxy coating was applied as it would be in the field, manually, or whether it was applied by factory spray equipment.

The EIS results for the polyurethanes and coal tar enamel coatings are presented in a similar fashion in figures 5.

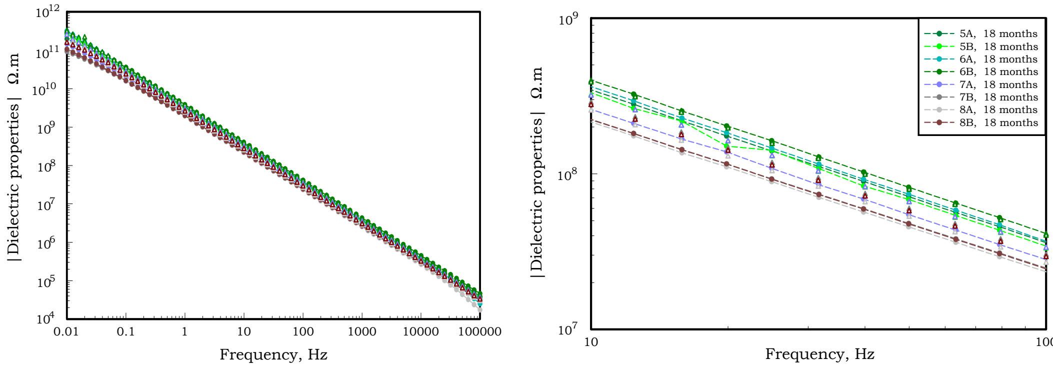


Figure 5. EIS impedance of polyurethane and coal tar enamel coatings; (a) the whole frequency spectrum, (b) shows an expanded view of a middle portion of the same data. In both diagrams the initial EIS spectra are depicted with unconnected triangle symbols and the results after 18 months immersion are depicted with circle symbols that are connected with a dashed line.

Polyurethane coatings, also, show no trend resulting from factory or manual application. Perhaps more importantly, the dielectric properties of both polyurethane and coal tar enamel coatings provide values that are approximately half an order of magnitude (the graphs have logarithmic axes) higher than for any of the epoxies. In addition, although there is some noise in the data at low frequencies, there is much less sign of a resistive plateau and very little change after 18 months immersion. Although a definitive conclusion might depend on more accurate measurement of thickness and longer term testing, the polyurethanes seem to give slightly superior performance in these tests than the coal tar enamel coatings.

The conclusion on the worth of the different materials can also be investigated using the Brasher-Kingsbury equation (8) to calculate a value for the amount of water entering the coatings during the immersion. The results are given in table 2. These values were calculated from the imaginary part of the impedance at 10^4 Hertz, which is a very common procedure. There is some scatter in the results that may be due to inaccuracies in measuring thickness, uneven thickness and other imperfections in the coatings, but in general the results are as one might anticipate. All of the epoxy coatings seem to absorb about 10% by volume of water during immersion. Within the scatter in the data, it seems that the polyurethane and coal tar enamel coatings imbibe almost no additional water upon immersion over this period. One must remember that using the Brasher-Kingsbury equation is approximate and measures the water that is additional to whatever amount was absorbed from ambient conditions before the test. It assumes that all the water has the same form as bulk water; that it is uniformly distributed and in pores that are spherical (on average).

Water absorption into polymer coatings is, of course, much more complex [Takeshita 2014] and some water will be associated in some form of bonded state with specific moieties on the polymer and some will be in a more-or-less bulk form in pores [Popineau 2005]. There has been considerable research but typically, it is believed that the Brasher-Kingsbury equation overestimates the amount of water in the coating [Philippe 2008]. Epoxies are more polar than many types of coatings and often take up 5-8% by weight of water whereas polyurethanes tend to absorb less water, but often contain ~2% of moisture under normal ambient conditions.

Table 2. Water content calculated from the initial and 18 month capacitances, (imaginary part of the impedance).

Sample	Capacitance Ratio	Water
Epoxy 1A	1.5_2	0.09_5
Epoxy 1B	1.5_5	0.10
Epoxy 2A	1.4_9	0.09_1
Epoxy 2B	1.4_2	0.08_1
Epoxy 3A	1.3_0	0.06
Epoxy 3B	1.5_9	0.10
Epoxy 4A	1.2_3	0.04_7
Epoxy 4B	1.2_7	0.05_5
Polyurethane 5A	0.96_9	-0.00_7
Polyurethane 5B	1.0_5	0.01_2
Polyurethane 6A	0.98_8	-0.00_3
Polyurethane 6B	1.0_4	0.01_0
CTE 7A	1.0_4	0.00_9
CTE 7B	1.1_0	0.02_3
CTE 8A	1.0_6	0.01_4
CTE 8B	1.1_8	0.03_8

CONCLUSIONS

EIS measurement is a useful, non-destructive technique for assessing the suitability of barrier coatings for providing corrosion protection. It can be used in the field as well as in laboratories. EIS measures the resistance of a coating to the passage of water and can be used to estimate the amount that a coating has absorbed.

Although there was some noise in the data at low frequencies, where the impedance is extremely high, there was no evidence of corrosion occurring under these coatings even after 18 months continuous immersion. All the coatings tested, epoxy, polyurethane and coal tar enamel, show very good impedance performance and are likely to remain useful for extended periods. There seemed to be no difference in performance between manually sprayed or factory sprayed coatings.

The measured impedance was transformed, using the area tested and the coating thickness, into the dielectric properties of the coating so that the material properties could be examined, regardless of the coating thickness. For the epoxy coatings, the impedance is reduced after the immersion due to water uptake within the coatings, and the 100% epoxy seemed to fare slightly better than the 80% formulation. It would be difficult to draw any general conclusions about solids content and performance since performance depends greatly on all the formulation ingredients, not just the polymer components.

Both polyurethane coatings and coal tar enamel coatings had dielectric properties (from the impedance) that were significantly higher than the epoxies and implies superior barrier properties for these coatings. In addition, a simple calculation using the Brasher-Kingsbury relationship, suggests that neither polyurethane nor coal tar enamel coatings absorbed extra water during the immersion, in contrast to the epoxy coatings. Thus one might suggest, from these tests, that of modern coatings, polyurethanes are a more promising alternative than epoxies.

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Watertightness of CFRP Liners for Distressed Pipes

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Abstract

Hydrostatic pressure tests of prestressed concrete cylinder pipe (PCCP) with bands of broken wires and lined with carbon fiber reinforced polymer (CFRP) have shown that certain CFRP laminates used as liners for remediation of distressed PCCP may not be watertight. Such test indicated the need to investigate the watertightness of CFRP laminates without expensive hydrotesting of full lengths of CFRP-lined distressed pipe. To address this need, the authors developed a special watertightness pressure chamber that allows testing of curved CFRP laminates, similar in geometry to a segment of CFRP liner installed in a rigid pipe, to pressures up to 500 psi. The pressure chamber simulates the stress state in the CFRP liner installed inside a pipe degraded to the extent that it does not resist the radial motion of the CFRP liner within a window. A number of watertightness concepts were developed, and CFRP specimens were built and tested. The results of the watertightness tests are presented along with the provisions for watertightness of CFRP liners. The procedure for qualification of different laminates for watertightness is discussed.

BACKGROUND

In the past four years, a research program [1, 2], sponsored jointly by the Water Research Foundation, a number of utilities, and two carbon fiber reinforced polymer (CFRP) manufacturers, was undertaken by the authors to form the basis of the standard under development by the American Water Works Association (AWWA) for CFRP renewal and strengthening of prestressed concrete cylinder pipe (PCCP). Several hydrostatic pressure tests of PCCP lined with CFRP and with simulated distress in form of induced wire breaks, performed as a part of this research program, have shown that certain CFRP laminates used as liners for remediation of distressed PCCP may not be watertight [3, 4], and indicated the need to investigate the watertightness of CFRP laminates.

The cause of loss of watertightness is not failure of the carbon fibers, but either improper termination details or high transverse strains acting on the laminate, resulting in cracking of the resin between fiber bundles. Proper termination design is outside of the scope of this paper. Strain-induced transverse cracking is not strength related and typically occurs in laminates designed for strength. Hence watertightness is influenced by (a) brittleness of the resin, e.g., epoxy thickened with silica fume is more brittle than neat resin; (b) existence of strains high enough to cause system of transverse cracks in different layers that would compromise watertightness; (c) laminate design and number of layers; (d) existence of a layer in the laminate design with much higher strain capacity in the transverse direction; and (e) existence of impervious coating. Loss of watertightness causes leakage through the laminate. The leakage rate is highly pressure dependent, and is in form of beading drops at low pressures and beaming streams at high pressure. Testing watertightness of CFRP-lined full-scale steel pipe or PCCP with simulated distress by hydrostatic pressure testing is not feasible and very expensive to conduct. There is a need for a simpler test that can be used for qualification of laminates designed for CFRP renewal of distressed pipes.

This paper presents the results of a study undertaken to develop a practical way to test the watertightness of CFRP laminates intended as liners for renewal of distressed PCCP or steel pipes. The approach adopted in this paper is to design and build a pressure chamber to test a piece of the laminate simulating the stress state in the CFRP liner when the host pipe continues to degrade and can no longer provide support to the liner over its entire wall, as in hydrostatic pressure testing of the entire distressed pipe with CFRP liner.

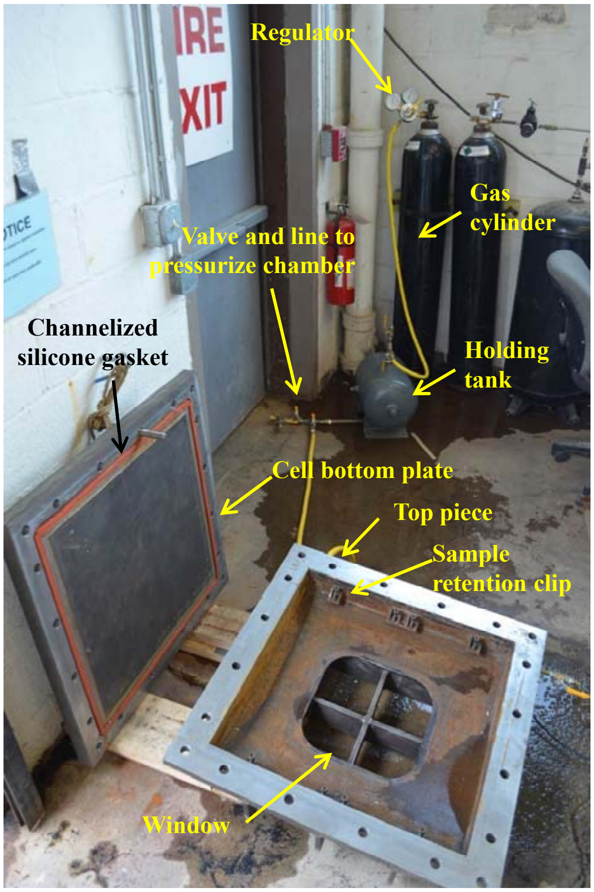
In the remainder of this paper, the design and construction of the pressure chamber and test specimens, test procedure, and some results of watertightness tests are presented along with conclusions and application recommendations.

TEST APPARATUS AND CONSTRUCTION OF SPECIMEN

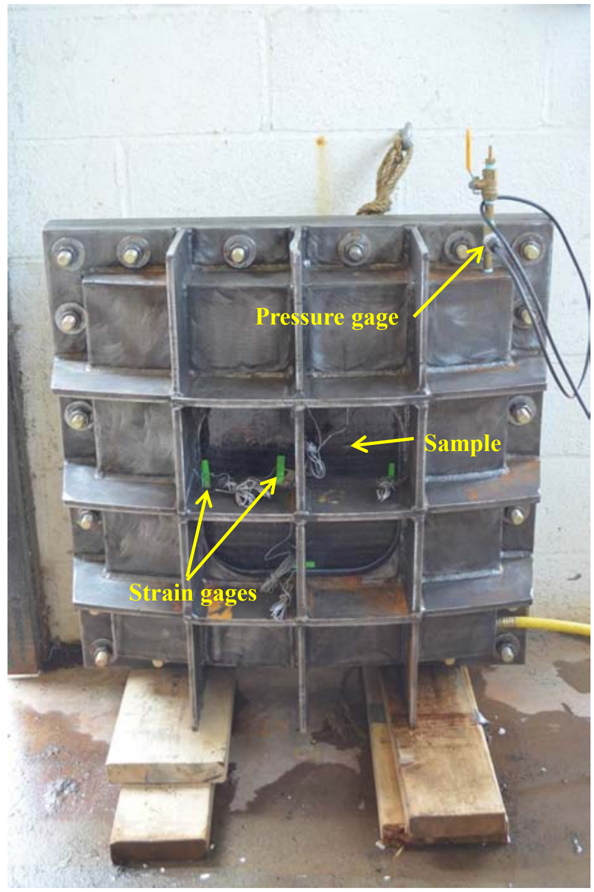
The test apparatus consists of a pressure chamber that is 24 in. square in plan, a high-pressure water holding tank, a compressed gas cylinder, and pressure gage and valves as described below. The apparatus is shown in Figure 1a. The pressure chamber is designed to resist a pressure of 500 psi and is fabricated from welded steel plates with stiffeners. The pressure chamber consists of two parts: a 24 in. by 24 in. rectangular top weldment consisting of a cylindrical plate with a 48 in. radius to simulate the inside surface of a 96 in. diameter steel pipe, and a 12 in. by 12 in. window through which the CFRP laminate is allowed to deflect. This top weldment is referred to as the top plate and has a machined bottom surface for bolting to the base plate. The base plate is a shallow box with open top. The mating surfaces of the two plates are machined, and a groove is carved in the base plate mating surface to capture a silicone gasket.

Test specimens are constructed on approximately 22 in. by 22 in., 16 ga steel sheet rolled to a radius of curvature of 48 in. in one direction to simulate a 96 in. diameter pipe curvature with a 12 in. x 12 in. window, and a removable curved plug sheet to allow construction of laminate. The steel sheets are sandblasted to an SSPC SP-10, near-white finish. Immediately after constructing the test laminate on the steel sheet, the specimens are allowed to cure at 73°F and 50% relative humidity to a minimum of 85% cure as determined by differential scanning calorimeter testing according to ASTM E2160. After curing, six unidirectional strain gauges are attached to the CFRP laminate in the window as shown in Figure 1b. Gauges A, B, 1, and 2 are 1 in., and Gages C and D are 0.25 in. long. The materials for the construction of test specimens presented here were obtained from two manufacturers of CFRP systems for buried pipes. Samples were constructed in the SGH laboratory with the exception of a few that were prepared by one of the manufacturers.

The strain gages attached to the test specimen (Figure 2) are waterproofed by coating them with epoxy. Then, the test specimen with strain gages is installed in the top plate, and the two plates are bolted together with twenty 3/4 in. in diameter high strength bolts.



(a)



(b)

Figure 1 – Test apparatus: (a) disassembled, (b) assembled. The window simulates the broken wire zone and allows outward deflection of CFRP laminate under pressure.

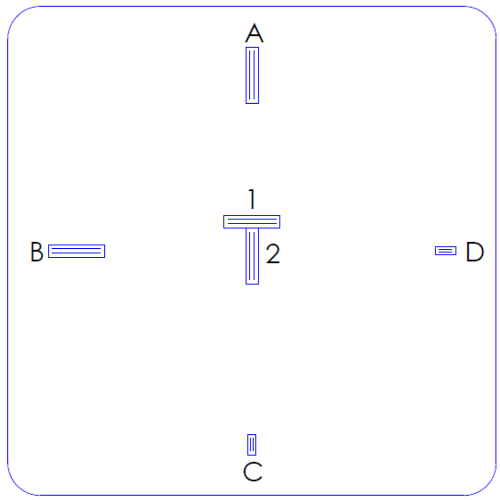


Figure 2 – Strain gauge layout.

To pressurize the pressure chamber, first the chamber and the holding tank are filled with water. The holding tank is then pressurized with air from the compressed gas cylinder. Pressure is controlled by a regulator at the gas cylinder and is also monitored with a pressure transducer at the top of the pressure chamber in line with the overflow valve. The pressure transducer is calibrated in the range of 0 to 500 psi.

A thin layer of open-cell foam and a 1/8 in. silicone rubber gasket are placed between the specimen and the steel box to seal the space between the specimen and the top weldment. The sample is held against the curved top plate of the cell and spans across the 12 in. square opening. The sample is restrained by angles that are pressed against the test specimen along the straight edges by bolts that press against clips welded on the interior of the cell wall. The water pressure further seals the sample against the curved surface of the top weldment.

The tests are conducted at or close to gage factor reference temperature of 24°C (75°F). No additional temperature correction is applied.

FINITE ELEMENT ANALYSIS OF CFRP TEST SPECIMENS

To study whether the simulated pressure test can simulate the stress state of the CFRP liner of full pipe in a hydrostatic pressure test, we developed a finite element model of the test specimen in the pressure chamber subjected to internal pressure. The model is a one-quarter model of a curved steel plate with a hole in the center overlaid with three layers of CFRP. The steel plate measures 4 ft-6 in. in the longitudinal direction, 5 ft-0 in. in the circumferential direction and has a radius of curvature of 48 in. The dimensions of the window are 12 in. x 12 in. The steel plate has a thickness of 0.25 in. and a modulus that would simulate the stiffness of test apparatus. The CFRP has a thickness of 0.08 in. per layer. The outer layer of CFRP is oriented with fibers running in the longitudinal direction, and the inner layer (on the wet side) of CFRP is oriented with the fibers running in the circumferential direction. The open-cell foam and gasket between the test specimen and the curved plate is modelled based on the results of compression tests performed on a 2 in. by 2 in. piece. Pressure loads of 100, 200, 300, 400, and 500 psi are applied to the inner layer of CFRP, and maximum membrane strains in each layer are determined.

The steel is modeled with a modulus of elasticity of 29,000 ksi and Poisson's ratio of 0.3. The properties of CFRP are based on measured mean tensile modulus of 12,370 ksi in the fiber direction and 910 ksi in the transverse direction, estimated shear modulus of 337 ksi, and a Poisson's ratio of $\nu_{LT} = 0.296$.

The model, the calculated deflections and strains in fiber and transverse directions of different layers at 500 psi are shown in Figures 3 through 5. Figure 3 shows the deformed shape. Figures 4 and 5 show the maximum strains in the fiber direction. The strain in the transverse direction of the inner layer is higher; such a high strain may cause cracking between fiber bundles, but does not cause rupture of laminate.

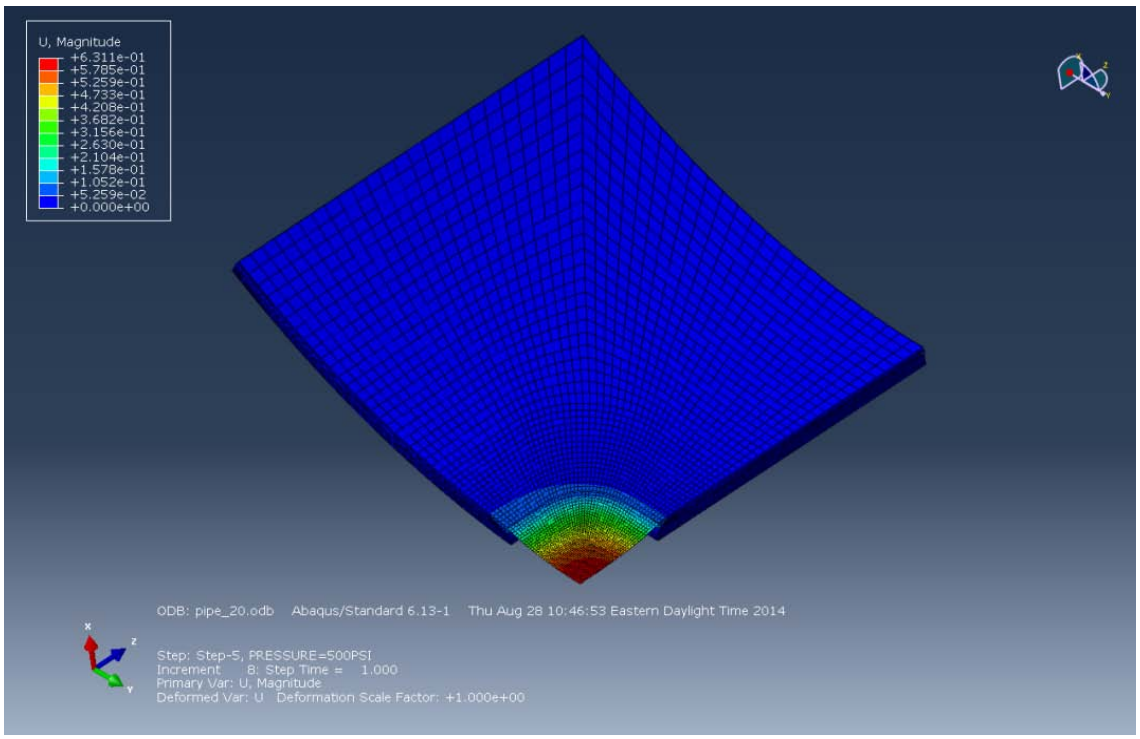


Figure 3 – Deformation of CFRP at 500 psi pressure. Note maximum deflection of 0.63 in.

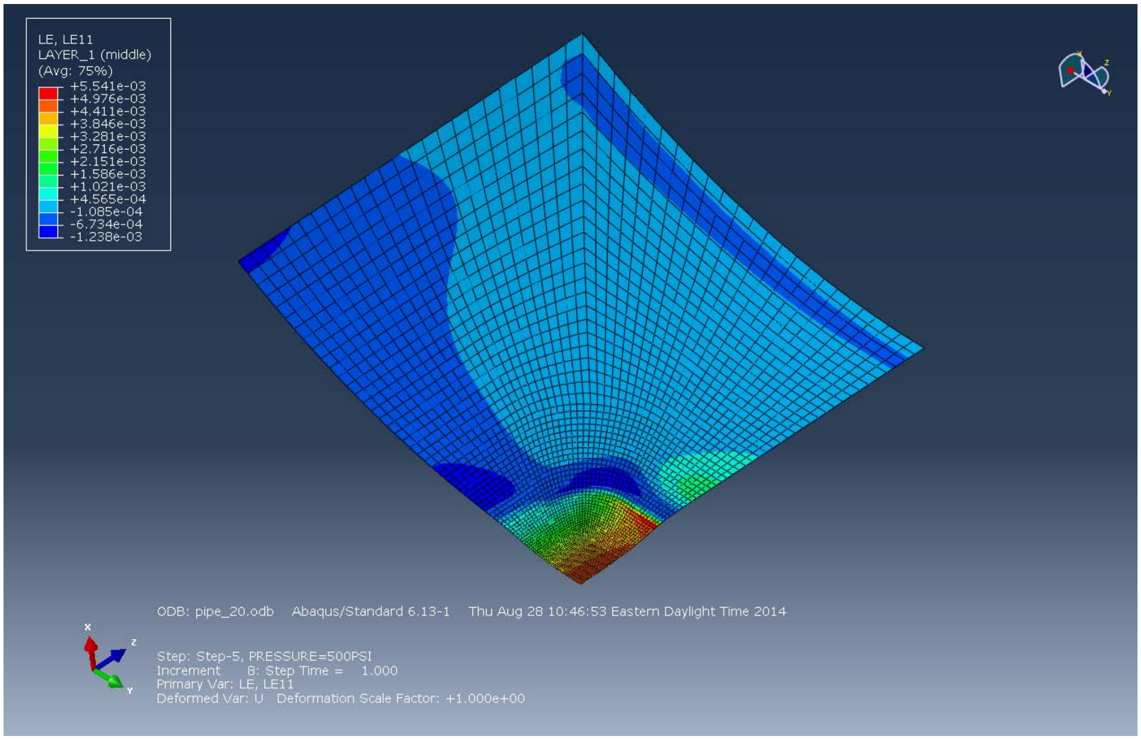


Figure 4 – Strains in the fiber direction of the inner layer (fibers run in the curved direction) at 500 psi pressure. Note maximum strain of 0.55%.

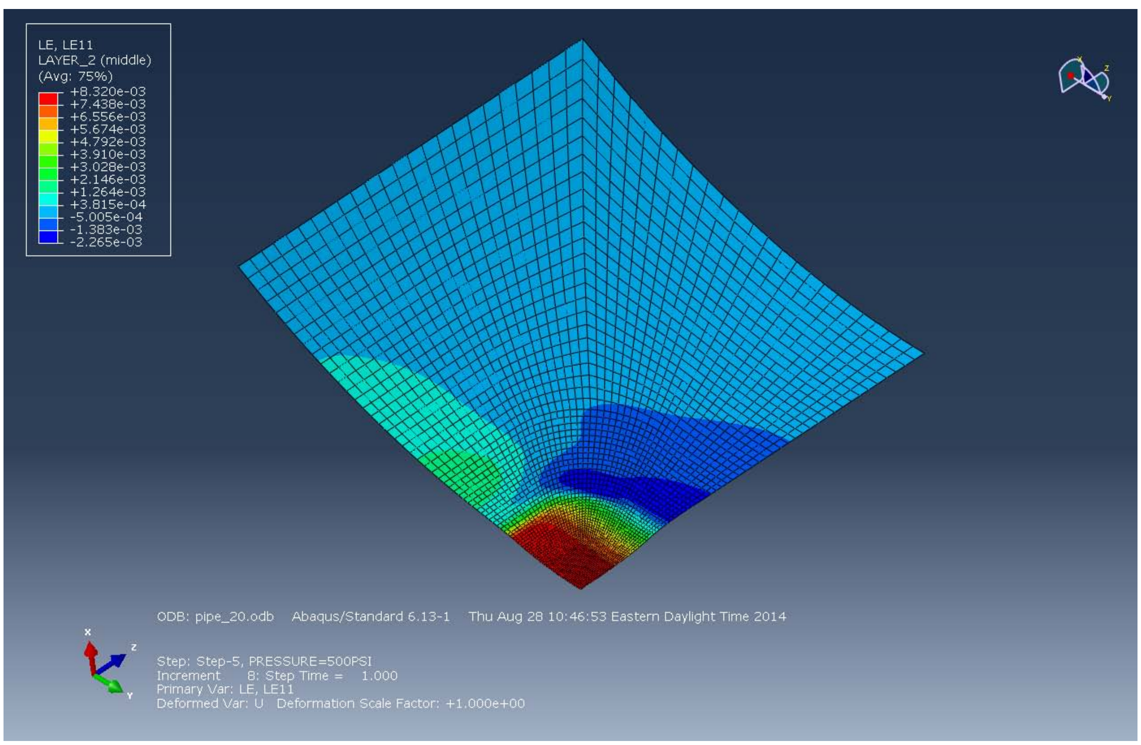


Figure 5 – Strains in the fiber direction of the outer layer (fibers run in the straight direction) at 500 psi pressure. Note maximum strain of 0.83%.

WATERTIGHTNESS TESTING PROCEDURE

The following summarizes our procedure for conducting each test.

a) Place a silicone gasket and open-cell foam gasket on the curved surface of the top plate of the pressure chamber, where the test specimen will be placed. Place the silicone gaskets onto the base plate channel using spray adhesive.

b) Seat the specimen on the curved surface of the top plate of the pressure chamber.

c) Connect the top plate and base plate using 3/4 in. high-strength bolts.

d) Attach the pressure transducer and strain gages to the data acquisition system and mount the dial gauge to measure specimen deflection.

e) Zero the strain gauges.

f) Fill the pressure chamber with water and allow it to reach house pressure of approximately 60 to 80 psi.

g) Apply pressure in 25 psi increments and record deflections until loss of watertightness or rupture of the specimen occurs. At each increment of pressure, examine the CFRP laminate for leakage and find the source of leakage through the laminate.

h) Record pressure and strain at a rate of 2 Hz using data acquisition system.

WATERTIGHTNESS TEST RESULTS

We performed a series of tests on CFRP laminates using materials from different manufacturers with and without additional layers incorporated in the laminate architecture for watertightness, such as proprietary glass fabrics, thickened epoxy coating, and polyurethane coatings.

The results of nine of these tests, including the source of materials, laminate architecture, maximum pressure before leakage, and the maximum measured strain are summarized in Table 1. In the table, “L” and “H” indicate CFRP layers in the longitudinal and hoop directions, respectively, and “G” indicates GFRP layer.

Table 1 – Summary of test results

Specimen	Description	Materials	Pressure at Leak	Max. Strain
1	1L+1H	CFRP and resin proprietary products of Manufacturer 1	251 psi	0.54%
2	1L+1G+1H G = thin layer of woven glass fabric	CFRP and resin and glass fabric proprietary products of Manufacturer 1	400 psi	0.78%
3	1L+1H+ topcoat of polyurethane	CFRP, resin type A, and polyurethane coating are proprietary products of Manufacturer 2	375 psi	0.50%
4	1L+3H+ topcoat of polyurethane	CFRP, resin type A, and polyurethane coating are proprietary products of Manufacturer 2	403 psi	0.64%
5	1L+3H + topcoat of thickened epoxy	CFRP, resin type A, and Cab-o-Sil thickened epoxy are proprietary products of Manufacturer 2	150 psi	0.35%
6	1L+1G+1H G = 10 mil woven glass fabric	CFRP, resin type B, and woven glass fabric are proprietary products of Manufacturer 2	250 psi	0.60%
7	1H+1L+1L+1H	CFRP and resin type A are proprietary products of Manufacturer 2	125 psi (Water beaded on CFRP surface at multiple locations at 150 psi)	0.24%
8	1H+1L+1G+1L+1H G = 34 mil bidirectional stitched glass fabric layer	CFRP, resin type A, and the bidirectional stitched glass fabric are proprietary products of Manufacturer 2	350 psi	0.48%
9	1G+1H+1L+1H G = 14 mil woven glass fabric	CFRP, resin type A, and the bidirectional woven glass fabric are proprietary products of Manufacturer 2	81 psi	–

The test results show the following:

- The test specimen with a laminate design consisting of one longitudinal layer and one circumferential layer made of CFRP from Manufacturer 1 failed at 250 psi. With the addition of the manufacturer's proprietary woven glass fabric embedded in the laminate, the pressure reached 400 psi before losing watertightness with the maximum strain reaching 0.78%. Somewhat smaller pressures were obtained for a similar laminate with woven glass layer using the products of Manufacturer 2. The tests show that bidirectional glass fabric may be used for watertightness of CFRP laminates. The pressure achieved in the test before loss of watertightness depends on the type of glass fabric used.
- The test specimen with a laminate design consisting of one longitudinal layer and one circumferential layer of CFRP and a proprietary polyurethane

coating remained watertight up to 375 psi where the maximum strains measured were at about 0.50%. The laminate made with one layer of longitudinal and three layers of circumferential CFRP using the same materials plus the same coating remained watertight up to 403 psi where the maximum strains measured were at about 0.64%. The tests indicate that special coating can be incorporated into the design or applied to a constructed liner to ensure watertightness.

- The test specimen with one layer of longitudinal and three layers of circumferential CFRP and a coating of thickened epoxy lost watertightness at 150 psi where the maximum strains measured were at about 0.35%. The test indicates that thickened epoxy alone cannot be relied upon for watertightness.
- The test specimen with a laminate design consisting of a symmetric laminate with two longitudinal layers and two circumferential layers of CFRP lost watertightness at 150 psi where the maximum strains measured were at about 0.24%. The laminate made at the same time with the same materials and an additional bidirectional glass layer reached 350 psi and the strain reached 0.48% before loss of watertightness. The tests show that addition of a proprietary bidirectional woven glass fabric can provide a watertight laminate up to 350 psi.
- A laminate made with one 14 mil thick woven glass fabric layer applied to the steel substrate, and one longitudinal layer and two circumferential layer of CFRP failed by debonding of the glass layer and loss of watertightness at pressure of 81 psi. This test indicates that the woven glass layer applied to the steel substrate cannot ensure watertightness of laminate.
- Further testing is needed to determine whether a relationship can be established between strain levels and watertightness for specific laminates.

ACCEPTANCE CRITERIA

The watertightness test results in bending of the laminate in both directions at the edges of the window and membrane strains away from the boundaries. However, the area that is subjected to maximum stress is limited. As a result of this effect, there may be some scatter in the data obtained, and the measured maximum pressure before loss of watertightness in a single test may be an upper bound. For this reason, it is prudent to consider multiple tests (three or more) and select the least of maximum pressures measured as the pressure corresponding to the point of loss of watertightness, apply a reduction factor of 0.5 to this pressure, and require that the resulting pressure exceed the maximum working plus transient pressure in the pipeline.

The results obtained from watertightness testing are applicable to pipes having the same diameter as the test pipe. For different diameter pipe, the same maximum strain must be maintained, which is equal to the combined membrane and bending strains. The membrane and bending strains for the same CFRP laminate are proportional to the radius. Therefore, the pressure at which the watertightness is compromised, P_{WTL}, may be considered proportional to the hoop strain, or

$$(P_{WTL})_{pipe} = (P_{WTL})_{test} \frac{R_{test}}{R_{pipe}} \frac{(\sum E_H t_H)_{pipe}}{(\sum E_H t_H)_{test}}$$

where R = radius, E_H = hoop strain, t_H = hoop layer thickness, subscript "test" refers to the watertightness test, and subscript pipe refers to the CFRP liner.

CONCLUSIONS AND RECOMMENDATIONS

From the test results, we can make the following conclusions and recommendations:

1. The watertightness of a CFRP laminate to be used as a liner in a pipe must be established by either full-scale hydrostatic pressure testing of a CFRP-lined pipe or by simulated pressure test as presented here.
2. A CFRP laminate that is not watertight can be made watertight by introducing in the laminate design an additional impervious layer made of glass fibers or of other materials, a coating layer applied to the inside surface of the laminate, or additional CFRP layers.
3. Introduction of a certain glass layer in the CFRP laminate architecture can improve watertightness of the laminates. The extent of improvement depends on the properties of CFRP, glass, and layup sequence.
4. Polyurethane coating can substantially improve the watertightness of a laminate.
5. Thickened epoxy alone cannot ensure watertightness.
6. Application of a glass fabric as the first layer on the pipe surface prior to CFRP application does not provide watertightness as the glass layer delaminates from the CFRP laminate under pressure.
7. A laminate is watertight if it incorporates a number of layers and sequence that is proven to be watertight by testing, i.e., addition of other layers on either sides of a watertight laminate does not compromise its watertightness.
8. Available watertightness test results may be applied to the design of CFRP liners, if the CFRP laminate tested has remained watertight at a pressure of at least twice the maximum working plus transient pressure used in the design of CFRP laminate.
9. Further research is needed to determine whether a relationship can be established between strain levels and watertightness for specific laminates, and whether pressure at loss of watertightness can be predicted analytically.

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Oil and Gas Pipeline Technology Finds Uses in the Water and Wastewater Industry

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Abstract

Failure in oil and gas pipelines due to leaks has led regulators to require operators to implement ever more rigorous inspections. However, advances in inspection technology developed for oil and gas pipelines have not been fully utilized for water and wastewater pipelines. ANSI/NACE Standard Practice 0502 – Pipeline External Corrosion Direct Assessment Methodology has been developed to ensure safe operation of pipelines and prevention of external corrosion in non-piggable pipelines. This standard requires a minimum of two indirect inspections to confirm the most susceptible locations on a pipeline for external corrosion to occur. While legacy technology requires a technician to first locate and map a pipeline, then to conduct individual inspections for coating faults, cathodic protection, and soil data, external line inspection (XLI) technology combines up to 10 different inspection techniques into one integrated inspection. A case study is provided to show the potential and limitations of this advanced inspection technology.

INTRODUCTION

Due to the high consequences of corrosion and leaks in underground pipelines, External Corrosion Direct Assessment (ECDA), as described in ANSI/NACE SP0502 [1], was developed in an attempt to proactively prevent external corrosion and ensure integrity of oil and gas pipelines. ECDA is a continuous improvement process intended to identify and address locations at which corrosion activity has occurred, is occurring, or might occur. For instance, ECDA identifies areas where coating defects have already formed, and can ascertain where cathodic protection is insufficient and corrosion is possible, before major repairs are required.

ECDA is a four-step process that includes pre-assessment, indirect inspection, direct examination, and post-assessment. Pre-assessment requires the integration of historical, construction, and maintenance records. Indirect inspection leads to the selection of at least two complementary indirect inspection tools be used to assess coating conductance and cathodic protection. Following indirect inspection, direct examinations are performed at likely locations to contain coating or corrosion damage. Direct examination prioritizes the findings of indirect inspections and involves excavation (in underground pipelines) of locations where coating flaws and corrosion are most likely, measurement of coating damage and corrosion defects, evaluation of damage severity, root cause analysis, and overall evaluation. The ECDA concludes with a post assessment that defines and/or determines reassessment interval and evaluates the effectiveness of the ECDA.

The following aboveground inspections may be used to perform an ECDA: direct current voltage gradient (DCVG), alternating current voltage gradient (ACVG), cathodic protection close interval potential survey (CP CIPS), alternating current—current attenuation (ACCA), side drain surveys (bare or ineffective coated pipelines). Normally these aboveground inspections are used in conjunction with pipe locating, soil resistivity measurements, and Global Positioning System (GPS) surveys. Each inspection and survey technique is typically conducted independently; the line is located and marked with an electromagnetic locator, then each mark is recorded by GPS, and then the coating and CP inspections are conducted. The data recorded from these surveys and inspections, as well as soil chemistry and resistivity data are all subsequently combined, usually in the office post survey.

The indirect inspection step of the ECDA can be labour intensive and costly. Also, the reliability of the pipeline integrity data collected depends on the inspection equipment used and the qualifications and experience of the field technicians. Faced with these challenges, XLI technology and supportive XLI software were developed for comprehensive aboveground surveys and integrity data analysis. XLI technology can combine all the above mentioned inspections in one pass. For instance, CP CIPS, DCVG, ACVG, ACCA, GPS/Geographic Information System (GIS), and soil resistivity measurements and a depth of cover (DOC) survey can be done in one integrated inspection. This technology is currently used in oil and gas pipeline inspection in North America and potential for water pipelines in the wastewater sector. XLI technology brings huge improvements to integrity assessment, increasing reliability while reducing time and costs to collect, process, analyze, and report inspection results. Whereas legacy technology requires locating and mapping of a pipeline, and individual inspections for coating faults, cathodic protection, and soil data, XLI technology integrates up to 10 different inspection techniques in one inspection.

XLI technology is significant to utilities for the following reasons: (1) Data regarding soil properties, depth of cover, and cathodic protection assessments can play an important role in evaluating a pipe's condition and assessing the risk of failure. (2) Data collected is GIS referenced, mapping the pipeline, and can be integrated and spatially viewed on a GIS platform. Aboveground surveys described below can be performed in one integrated XLI inspection.

Cathodic Protection Close Interval Potential Survey (CP CIPS)

It is generally accepted that when two complementary technologies are applied simultaneously to corrosion control, the results will be better than when the two technologies are applied individually. This is exactly what coating and cathodic protection do when used together. When cathodic protection and coatings are used together in a pipeline, the cathodic protection system can provide protection at holidays in the coating.

The conjunction of coating and cathodic protection is the most effective way of controlling corrosion of underground steel pipelines, but both are subject to failures. Coatings have holidays (breaks and defects in the coating film) where the pipeline metal can be exposed to corrosive environments and cathodic protection can reduce or prevent significant corrosion. Like the coatings having holidays, the CP can be shielded or affected by stray currents rendering it ineffective. Both coating holidays and cathodic shielding can lead to corrosion, and therefore their effectiveness must be verified through inspections to ensure pipeline integrity. Also, a high current demand on bare pipe can lead to attenuation of the cathodic potential.

CP is an effective technique to mitigate the corrosion of underground or underwater steel pipelines. CP simply involves applying a direct current (DC) potential between an anode (a sacrificial metal) and a cathode (the metal needing protection). The resulting current must flow from the anode through a surrounding electrolyte (which could be water or soil) to the surface of the pipeline (the cathode). Corrosion can be mitigated by negatively polarizing the pipeline to a certain potential [2]. By polarizing all the cathodic sites to the most negative potentials of the anodic sites on a pipeline receiving cathodic protection, there would be no driving for corrosion and external corrosion of the pipeline is mitigated.

A CP CIPS is and inspection technique performed from above ground to assess the effectiveness of cathodic protection on buried pipelines. In the past, the assessment of cathodic protection by pipeline operators has relied on CP surveys taken on test posts located approximately every mile, but CP surveys on test posts do not guarantee cathodic protection of the entire pipeline because there could be unprotected areas between test posts. CP CIPS is the most effective way to assess cathodic protection of the entire pipeline in close intervals (typically every 3'-10'). Performing CP CIPS with XLI technology requires corrosion surveyors to walk aboveground of the buried pipeline and take measurements at close intervals (less than 1.5 m) while simultaneously recording the rectifier ON and instant OFF pipe-to-soil potentials with exact distance, depth of cover (DOC), GPS coordinates (latitude, longitude, and elevation) as well as date, time, status of differential correction, the number of satellites, and the position dilution of precision (PDOP) value.

The CP CIPS should be performed on a regular basis in accordance with Industry Recommended Practices since a CP CIPS can help in identifying interference, shorted casings, areas of electrical or geological current shielding, contact with other metallic structures, and defective electrical isolations joints. It can pinpoint where corrosion could occur and allow pipeline operators to make integrity decisions. Although CP CIPS data from XLI technology can tell pipeline operators where cathodic protection is ineffective, CP CIPS data cannot estimate wall loss.

Direct Current Voltage Gradient (DCVG) Surveys

The DCVG technique is an aboveground inspection performed with XLI technology for assessing pipe coating performance. DCVG uses the interrupted direct current from the CP source and can identify when the cathodic protection current reaches or leaves the pipeline .The DCVG technique is also complementary with CP CIPS for assessing the overall effectiveness of coating and cathodic protection of the pipeline.

Detection of a coating anomaly using DCVG relies on exposed metal creating a low resistive path that would increase the current density around a coating anomaly leading to voltage gradients that can be detected aboveground. The benefit derived with XLI technology over legacy technology is that it enables pipe locating and mapping to be accomplished simultaneously, without interrupting the survey. XLI technology utilizes digital techniques to measure and record the voltage between the two electrodes when the rectifier is on and off to record GPS coordinates and the time of each reading. With legacy technology, the surveyor uses intuition to locate a coating anomaly and manually determine remote earth for post survey calculation and estimation of coating severity index. With XLI technology, the coating anomaly severity index (DCVG % IR) is digital, improving the reliability of the data and reducing the time and costs of collecting the data.

DCVG surveys with the latest XLI technology has proven numerous benefits over legacy technology with data reliability, auditing, time and cost saving for pipeline operators. DCVG effectiveness can still be reduced by the same shielding factors that prevent CP function, as well as the pipeline environment (soil resistivity, corrosion products), the depth of cover, and electrical interference.

Alternating Current Voltage Gradient (ACVG) Surveys

The ACVG is similar to the DCVG technique. Both rely on an exposed bare metal creating a low resistive path that would increase the current density around the coating anomaly location leading to a voltage gradient in the earth. The major difference is that the ACVG technique involves impressing an alternating current with an AC signal transmitter (SPECTRUM XLI Line Illuminator) between the pipe and the earth and detecting the voltage drop around the coating anomaly whereas DCVG technique uses impressed and interrupted direct current, usually from a cathode protection source(s).

An ACVG inspection can be affected by depth of cover and/or current and probe spacing. If these variables are not properly corrected, pipeline operators could incur huge capital costs excavating pipelines for coating repairs based on false ACVG. To address the effect of these variables on ACVG, XLI technology optimized coating anomaly detection and prioritization methodology [3] making XLI technology to detect and pinpoint coating anomalies with a high level of precision.

Although performing ACVG inspection using XLI technology can pinpoint coating anomaly with high level of precision, it can be affected by interference, depth of cover and pipeline environmental factors (soil resistivity, corrosion products).

Alternating Current—Current Attenuation (ACCA) Survey

While voltage gradient techniques (ACVG, DCVG) rely on soil contact with probes for aboveground detection of underground pipe coating anomalies, ACCA is a non-contact method of detection of coating anomalies based on loss of AC current. The loss of AC current injected on a pipeline decreases with the length of pipe and the increase in dielectric property of the coating. For instance, higher dielectric coatings (e.g., polyethylene tape coatings) attenuate AC current less than lower dielectric coatings (fusion-bonded-epoxy) if both are placed in the same soil. The rate of AC current attenuation is dependent on the coating, the resistivity of the soil, and the size of the pipeline. A polyethylene (PE) tape coating applied on a smaller diameter pipeline in a high resistance soil would attenuate less than a fusion-bonded-epoxy (FBE) coating applied on a larger diameter pipeline in a lower resistance soil. Since the presence of a coating anomaly would generate a low resistive path on the buried pipeline, an ACCA survey can be used to detect coating regions of high conductance.

With XLI technology, current attenuation surveys are conducted by inducing an AC current onto the pipeline with a SPECTRUM XLI line illuminator and surveying the resulting electromagnetic field from aboveground with a SPECTRUM XLI receiver, and/or SeekTech SR20 locator and/or radio detection (RD) current mapper receiver. The measurements produced by the receivers are logged on pocket PC utilizing SPECTRUM XLI software that works as a fully computerized system to log GPS position, depth of cover, and current. SPECTRUM XLI software provides the calculations necessary to check for coating faults.

Although ACCA can pinpoint regions of low coating quality on a pipeline, it has its own limitations. The current measurements produced by the receiver are determined from the electromagnetic field produced by the AC current flow through the pipe, and therefore distortion of the magnetic field results in imprecise current values while using ACCA technique. There are numerous common factors encountered on a right of way survey that can cause distortion of the electromagnetic field and resulting current measurements. The most common are foreign electromagnetic fields that cause stray currents, and pipe bends that distort the electromagnetic field on the pipe. Because of the possibility that current measurements may be affected by distortion of the electromagnetic field, high attenuation indications do not always indicate the presence of coating faults. The current measurements taken near bends in the pipe, near foreign conductors (parallel or perpendicular), at pipeline or cable crossings, at taps or tees, or near power lines are all affected and can cause localized high attenuation indications, that may not be associated with actual coating faults.

As the attenuation technique is limited by many common factors, XLI technology use DCVG and ACVG surveys to confirm whether individual coating faults are present in areas of high attenuation. ACVG and DCVG surveys are more sensitive to small coating faults than the ACCA method.

Soil Resistivity Measurements

The corrosiveness of the soil relative to the buried pipe can be assessed using soil resistivity measurements. Since soil resistivity around the pipeline can give indication on the corrosiveness, it can be argued that soil resistivity measurement should be included as a complementary inspection requirement during the indirect inspection step of the ECDA process. XLI technology integrates soil resistivity information alongside cathodic protection and coating data for comprehensive integrity assessment of pipelines. Although a soil resistivity measurement is not required during the indirect inspection step of the NACE ECDA process, XLI technology integrates soil resistivity information so that severity of corrosion on the subject pipeline can be assessed with greater accuracy and confidence. For instance, if two buried pipelines have polarized potentials of -650 mV/CSE, and one pipeline lies in a very highly conductive soil (soil resistivity ~ 200 Ω-cm) and the other pipeline lies in a lower conductive soil (soil resistivity ~ 20,000 Ω-cm), the propensity for corrosion in the highly conductive soil would be expected to be greater although both pipelines fail to meet the -850 mV/CSE set by NACE [2] to ensure good cathodic protection.

However, soil resistivity measurements can be affected if there are parallel underground metallic pipelines at close proximity to the subject pipeline.

Integration of Pipeline Integrity Data Using XLI Technology

XLI technology combines up to 10 different inspection techniques, giving the pipeline operator a comprehensive pipeline integrity assessment. Figure 1 demonstrates how XLI software can aid a pipeline integrity specialist in analyzing pipeline integrity data.

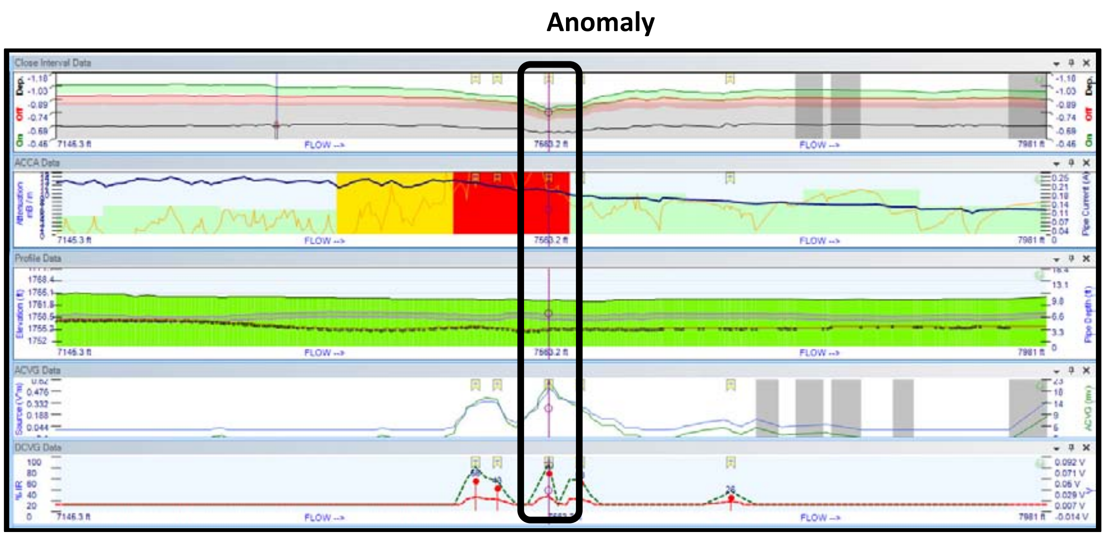

Figure1: Analysis of integrity data using XLI software.

The pipeline integrity data from the Anomaly area in Figure 1 are:

CP CIPS Instant Off = -730 mV/ CSE
CP CIPS On-Instant Off = 35.0 mV
ACVG norm = 510 mV*m
DCVG % IR = 71%
ACCA = 10 mB/m
No bends, sources of interference, or adverse conditions are present.

The pipeline integrity data in Figure 1 shows that the polarized potential failed to meet the -850m V/CSE NACE criterion to indicate good cathodic protection [2]. In the absence of depolarized survey data, this could indicate that the pipeline is not receiving the cathodic protection required to adequately mitigate corrosion or meet regulatory minimum specified by NACE [2]. The difference between Instant-On and Instant-Off (IR drop) is 35 mV. Although this is not a strict NACE requirement, such a small IR drop could represent low resistive path created as a result of a coating anomaly [4]. It could also represent interference from an uninterrupted rectifier source.

The DCVG IR is 71%. NACE SP0502-2008 [1] recommends an immediate repair for such a high DCVG. The amount of exposed metal could indicate that the coating anomaly is a major consumer of CP current and massive coating damage could be present. This could explain the sudden dip in the CP CIPS data from the adjacent areas. There is a sudden increase in the ACVG as shown in Figure 1, which could represent a low resistive path leading to sudden spike in voltage gradient. It can also be seen from Figure 1 (blue line in the red box) that the signal AC current declined gradually in the coating anomaly region while the attenuation rate increased to 10 mB/m. This could depict a reduced coating quality (increased coating conductance) in that region. It should be noted that while legacy technology uses DCVG or ACVG or ACCA for assessing the performance of coating, XLI uses DCVG, ACVG and ACCA giving comprehensive information for coating assessment as clearly depicted in Figure 1. Secondly, DCVG, ACCA and ACVG all indicated that massive coating anomaly could be present demonstrating how XLI technology can help pipeline operators pin point coating anomaly location with ease.

A case study presented in Figure 2 shows how different inspection results can be incorporated into an overall pipeline integrity assessment.

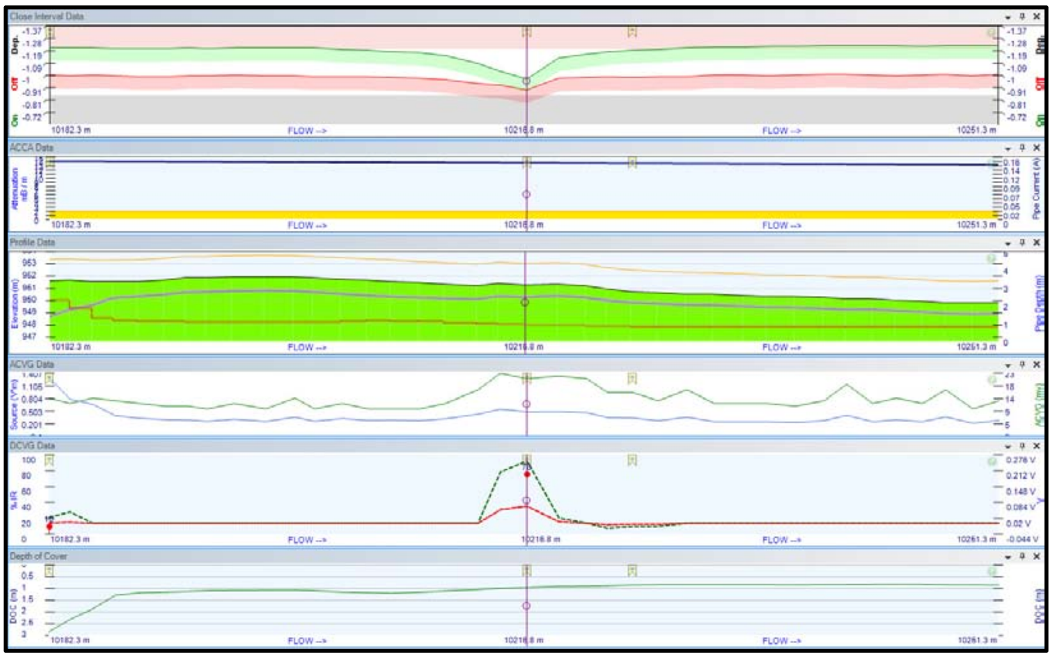

Figure 2: Analysis of integrity data using XLI software.

The case study in Figure 2 was taken from the indirect inspection step of the ECDA [1] performed using XLI technology. The subject pipe is 27 km long, 4in diameter, coated with Yellow Jacket®, and has been in operation since 1959. The indirect inspection involved CP CIPS, DCVG, ACVG, ACCA, DOC, GPS, and soil resistivity measurements.

The pipeline integrity data from Figure 2 is presented in Table 1.

Table 1: Pipeline Integrity Data from Figure 2

Virtual Distance (M)	Latitude (°)	Longitude (°)	CP CIPS On Pot (mV/CSE)	CP CIPS Inst off (mV/CSE)	Delta ON/Off Pot (mV)	DCVG % IR	ACVG (mV*m)	ACCA (mB/m)	Soil Res (Ω-cm)	DOC (m)
102216.8	51.153575	-113.55075	-972	-891	81	76	484	2	723.9	0.95

The pipeline integrity data show that the polarized potential is -891mV/CSE. According to NACE requirements [2], the pipeline should be receiving enough cathodic protection. The sudden drop in the polarized potential could represent exposed bare metal consuming much of the CP current. The IR drop of 81 mV could indicate a possible coating anomaly which is creating a low resistive path. The soil resistivity of 729.9 Ω-cm indicates that the soil could be highly conductive and thus corrosive [5] to the pipe. The DCVG IR of 76% indicates an immediate repair is recommended [1]. This could mean that coating damage could be present. It can also be depicted from Figure 2 that an increase in ACVG, dip in CP and increase in DCVG % IR all correlated very well using XLI supportive software. By considering the low soil resistivity around the pipe, the dip in CP, and DCVG and ACVG indications, a corrosion engineer would be able to make an informed pipeline integrity decision.

CONCLUSIONS

The authors believe that the capability of XLI technology to combine up to 10 different inspection techniques into one integrated pipeline inspection ensure improvement in reliability while reducing time and cost to collect data, and to process, analyze, and report inspection results.

It is generally known that when three complementary technologies are applied to coating inspection, the results would be better than the sum of the three technologies individually. This is the philosophy used in XLI technology for coating assessment. For instance with XLI technology, the integrity personnel is presented with DCVG, ACVG and ACCA coating assessment data making it a comprehensive coating assessment technology over the legacy technology that uses either DCVG or ACVG for coating assessment of an underground pipeline. This added advantage derived by using 3 different complementary coating assessment techniques is the drive for the use of this technology in oil gas and water sectors.

The complementary inspection techniques provided by XLI technology for assessing cathodic protection and coating on pipelines; would reduce huge necessity of excavations to explore suspicious indicators.

Data regarding soil properties, depth of cover and cathodic protection assessments can play an important role in evaluating the pipe's condition and assessing the risk and that is why all the three variables are integrals part of XLI technology. This demonstrate how XLI technology can provide pipeline operators with comprehensive integrity data to make informed decision and ensure better pipeline integrity management.

This advancement is currently used in oil and gas pipeline inspection in North America and a potential in the water and wastewater sector.

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Strategic Management of AC Pipe in Water Systems

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Abstract

Concerns about the durability and elevated consequence of failure associated with asbestos cement (AC) water main breaks has led utilities to increase focus on the management of AC water mains. This paper synthesizes results from the collaborative project between East Bay Municipal Utilities District (EBMUD), HDR, and the Water Research Foundation (WRF) titled "Development of an Effective Management Strategy for Asbestos Cement Pipe". This study has expanded water industry knowledge regarding AC pipe performance and recommended a:

- Method to quantify factors that drive pipe deterioration and useful life
- Appropriate level of renewal investment at EBMUD
- Condition assessment approach most useful in focusing renewal investments
- Method to identify and prioritize particular pipes for renewal

Note, these recommendations are dependent on utility specific variables including but not limited to desired level of service, cost of service constraints, geographic influences, loading influences, and construction practices. While the conclusions may vary by utility, the approach to answering these questions is meant to support all utilities in the development of effective management strategy of the AC pipe they own.

BACKGROUND

In the United States, AC water mains represent approximately eleven percent (Folkman, 2012) to fifteen percent (AWWA, 2012) of existing water main infrastructure. Concerns about the durability and elevated consequence of failure associated with AC water main breaks has led utilities to increase focus on the management of AC water mains.

For example, East Bay Municipal Utilities District (EBMUD) became increasingly concerned about its AC pipe inventory (~1,120 miles) in 2008 when a large spike in AC breaks occurred. Samples were extracted for phenolphthalein stain testing (Stain Testing), which confirmed that considerable degradation of the material had occurred and seemed to indicate a short remaining life. As shown in Figure 1, recent break history had shown a sharp increase in the break rate of older AC pipes (60 to 70 years old). Although only a small portion of the inventory was this old, over the next 20 years, approximately half of the inventory would reach this threshold.

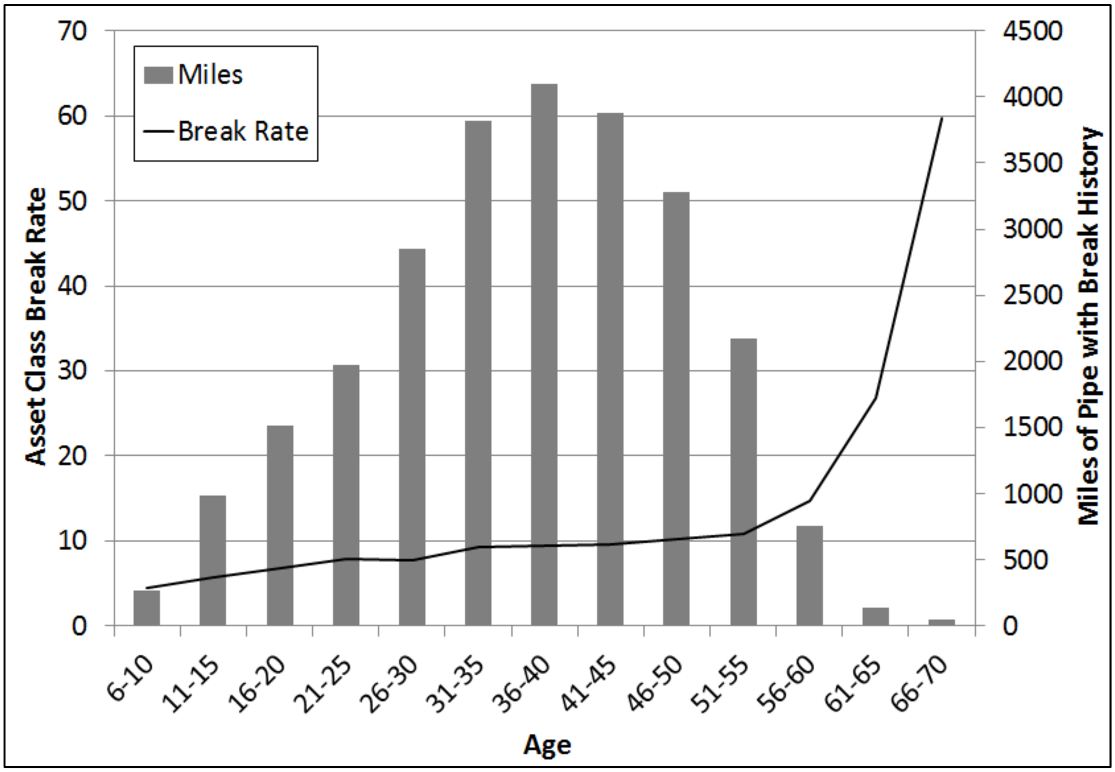


Figure 1. A large portion of the AC system was approaching an age at which the break rate was rapidly accelerateing (60-70 years old).

Given the heightened awareness regarding strategic management of AC water mains, the WRF partnered with EBMUD and HDR to fund a study to better understand the causes of AC water main breaks and how best to manage them. This effort culminated in the WRF Project 4480 titled “Development of an Effective Management Strategy for Asbestos Cement Pipe”. The objective of this report was to develop an effective AC water main strategy by answering the following questions:

- What factors drive classes of AC pipe to deteriorate at varying rates?
- What is the appropriate level of renewal investment?
- What factors should be used to identify and prioritize particular pipes for renewal?
- Which condition assessment tests are most useful in focusing renewal investments?
- How beneficial would a change in water quality be in prolonging AC pipe life?
- Would lining or sealing of AC pipe be a worthwhile investment?

This paper synthesizes the results of the first four bullets and is organized by each of the bulleted questions above. Note, the answers to some of these questions are dependent on utility specific variables including but not limited to desired level of service, cost of service constraints, geographic influences, loading influences, and construction practices. While the answers to these questions may vary by utility, the approach to answering these questions is meant to support all utilities in the development of effective management strategy of the AC pipe they own.

WHAT FACTORS DRIVE CLASSES OF AC PIPE TO DETERIORATE AT VARYING RATES?

The EBMUD system has characteristics that made this study of AC pipe performance particularly beneficial to the water industry. The EBMUD AC pipe inventory is large (more than 1100 miles) and was installed over the course of many decades. Their GIS system contains more than 23 years of good-quality break data. Most importantly, this pipe has been subjected to a variety of well-defined conditions, including varying pressures, soils, topography, water qualities, and climates. By examining differences in break performance, the influences of these factors could be observed. For the purposes of this study, pipe deterioration rates were measured as a function of infrastructure age verses break rate (annual breaks per 100 miles of pipe in service).

Nine factors (numbered below) were analyzed to assess whether they drove deterioration rates. Ground slope (#1), source water aggressiveness (#2), concrete corrosion potential (#3), and changes in system operations (#4) had insignificant or inconclusive factors. Age (#5) and diameter (#6) were significant factors but not primary drivers for performance. The three factors below were found to have the strongest influence on break-rate performance, as the AC pipes aged:

- **Material type (#7)**. Figure 2 shows that the performance of pipe installed after 1950 (Type II) is relatively homogeneous with respect to installation date. That is, while break rates increase with age, at a particular age, performance is similar regardless of the installation date. However, pipe installed prior to 1950 (i.e. Type I) breaks at a

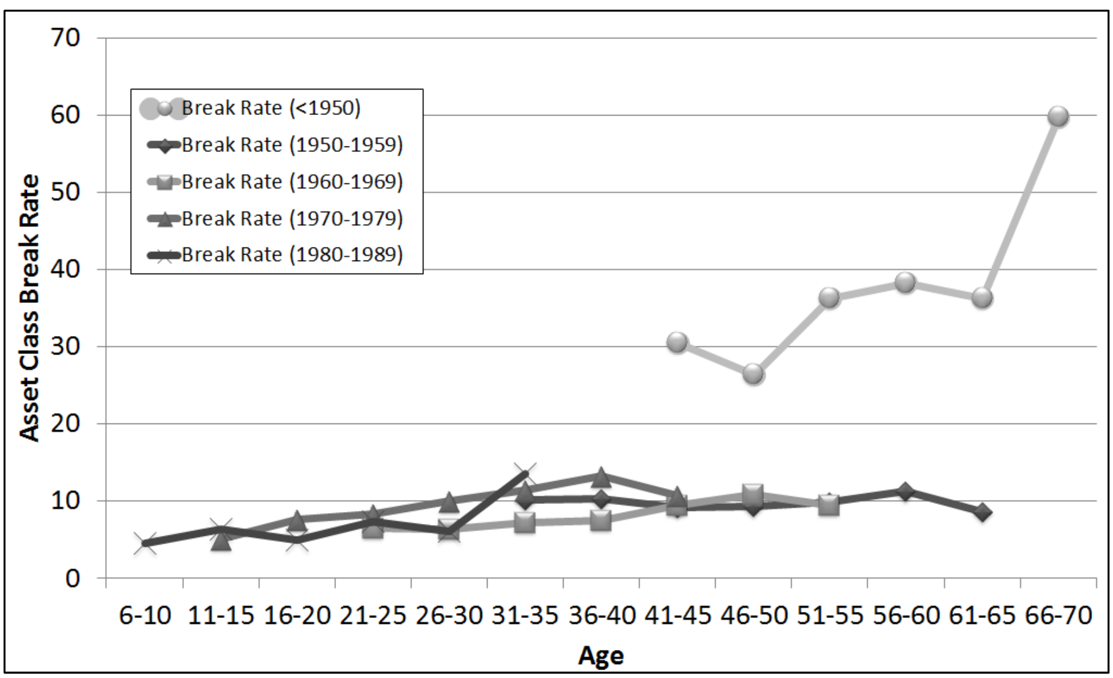


Figure 2. Analysis allayed fears that the vast majority of the AC system (installed after 1950) will expereince a rapid acceleration of break rates over the next 20 years.

much higher rate than Type II pipe, even at the same age. This is good news for EBMUD as approximately 98% of the system is Type II and is not expected to experience a rapid acceleration of break rates in the near future.

- **System pressures (#8)**. Figure 3 shows the impact of pressure on the deterioration of Type II pipe. Over the first 25 years, pressure has no discernable impact on performance. However, as they continue to age, the impact of higher pressure is felt and pipes subjected to higher pressures experience higher break rates.

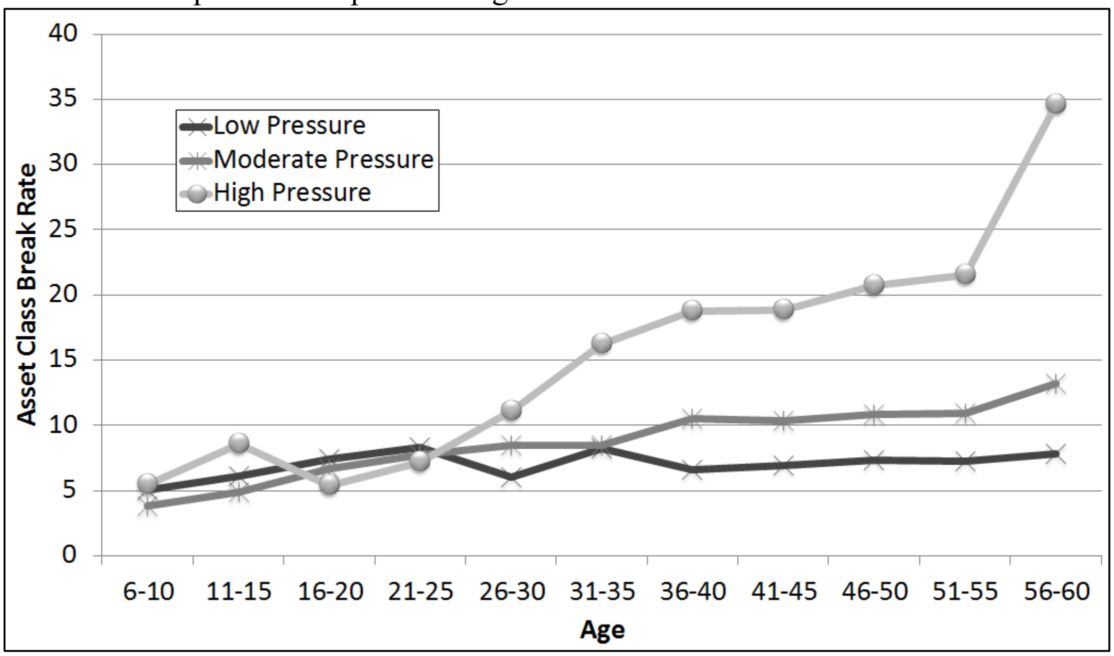


Figure 3. While negligible over the first 25 years of life, the impact of higher pressures eventually leads to higher break rates and shorter useful lives.

- **Shrink-swell (SS) potential (#9)**. SS potential estimates the cyclical stresses placed on pipes when the surrounding soils expand and contract. The severity of SS potential is dependent upon the linear extensibility (LE) of surrounding soils and the relative variation in soil moisture content. In the EBMUD system, soil moisture contents can be estimated based on location. Pipes west of the Oakland-Berkeley Hills (West) experience less variation in moisture content because of higher humidity, flatter topography, lower depth to groundwater, and more temperate conditions keep soils relatively moist year-round. East of the Oakland-Berkeley Hills (East), soil moisture content varies significantly depending on the season. Figure 4 shows the impact of shrink-swell (SS) potential on Type II pipe. Pipe exposed to high SS potential (i.e. pipes in the East with high or medium LE) deteriorated much faster than pipes exposed to Low SS potential.

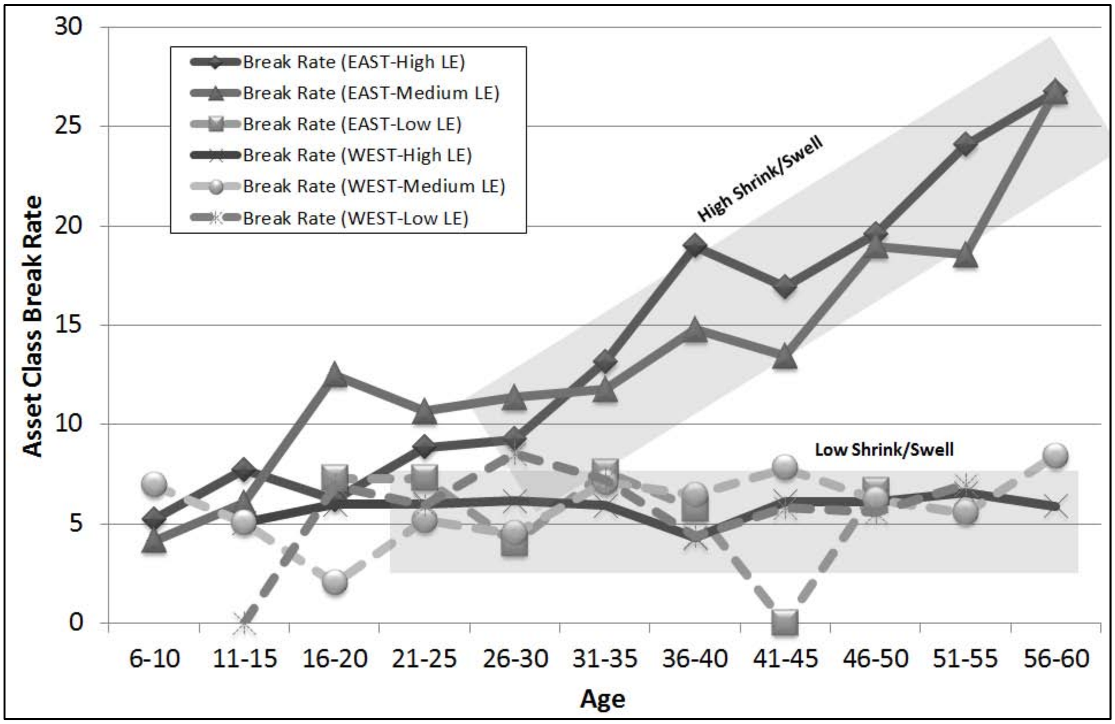


Figure 4. Pipes exposed to High SS potential deteriorate faster than pipes exposed to Low SS potential.

WHAT IS THE APPROPRIATE LEVEL OF RENEWAL INVESTMENT?

Based on the analysis described above, three factors (material type, pressure, and SS potential) resulted in twelve unique combinations of those factors. Several of these combinations had similar performance and were grouped to define four asset classes for the purpose of estimating useful life:

- Type I and Type II, High SS, High or Moderate Pressure
- Type II, Low SS, High or Moderate Pressure
- Type II, Low SS, Low Pressure
- Type II, High SS, Low Pressure

The Weibull Distribution was used to model two definitions of failure (DoFs) based on break history between 1990 and 2013 and pipe length (50-500 feet is "short pipe" while pipe greater than 500 feet is "long pipe":

- DoF 1: two breaks on short pipe and three breaks on long pipe
- DoF 2: three breaks on short pipe and four breaks on long pipe

The Weibull Distributions were then applied to the system installation profile to estimate the number of miles that will fail in each of the next 50 years. In the EBMUD system the average pipe length is 499 feet. However, in many cases, it is prudent to extend the boundaries of a replacement project to limit disruptions (e.g. customer, traffic, etc.), limit unit costs, replace nearby suspect pipe, and limit other social and political impacts (e.g. don't dig up a street twice within several years). Therefore, cost effectively replacing a certain length of pipe commonly requires replacing some pipe that has not yet failed. The average length of EBMUD condition-based replacement projects in 2012 was 1,360 feet. Therefore, it was assumed that for every 499 feet of pipe projected to fail by the model, an additional 861 feet of unfailed pipe would require replacement. Replacing this additional pipe will reduce renewal needs over the planning horizon (50 years) by some unknown percentage. This percentage is called the Project Packaging Percentage. Three project packaging scenarios (25%, 50%, and 75%) were modeled to determine how this assumption would impact investment levels.

Figure 5 summarizes the projected 50-year renewal investment need. The solid blue and green lines summarize the projected renewal needed to sustain the current backlog of failed pipe based on DoF1 and DoF2 respectively based on how optimistic the model assumption is in relation to the project packaging percentage. The dashed orange line represents EBMUD's historic renewal level. The dashed red line represents the Draft 10-to-40 Plan which is a proposed investment level being consider by EBMUD.

The analysis shows that through 2020, the 10-to-40 Plan will roughly keep pace with the AC pipe failures. Between 2030 and 2050, the 10-to-40 Plan may exceed the replacement level required to maintain break rates. Therefore, this report recommends that EBMUD should:

- Follow the 10-to-40 Plan through the 2020 timeframe
- Reevaluate long term renewal needs prior to 2020 to refine the appropriate renewal investment needs

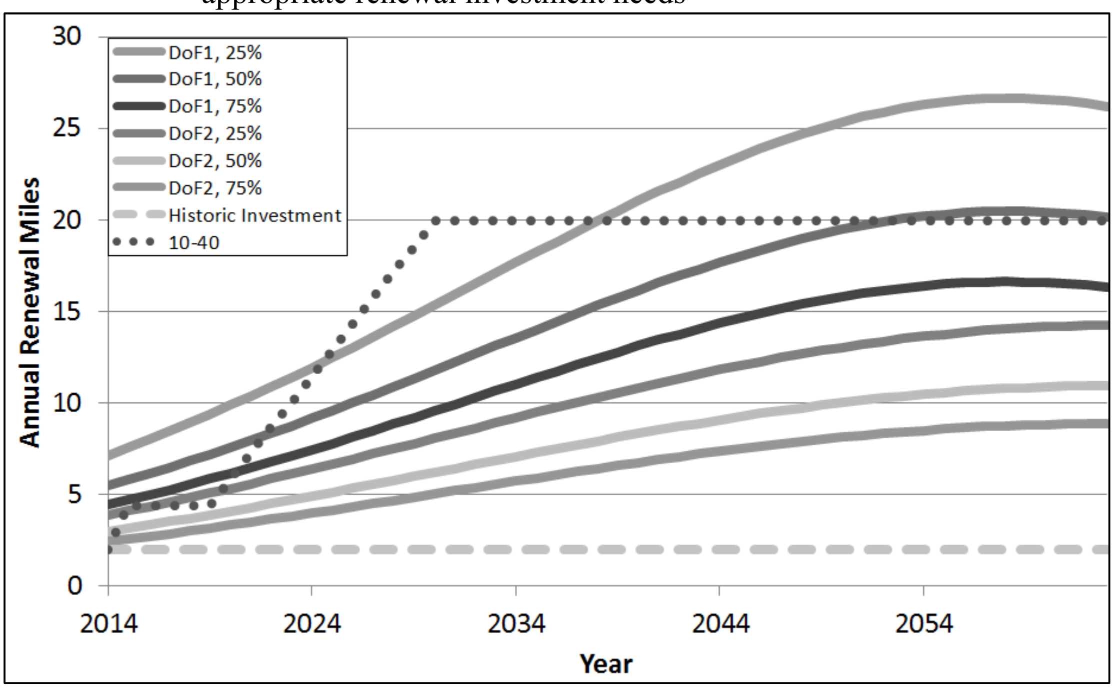


Figure 5. Data driven renewal investment projections confirm that EBMUD should increase near term renewal investments.

WHAT FACTORS SHOULD BE USED TO IDENTIFY AND PRIORITIZE PARTICULAR PIPES FOR RENEWAL?

Figure 6 summarizes the relationship between historic break count and the duration until the next break. So for example, the average duration between the first and second break is 5.5 years while the average duration between the second and third break is 4.7 years. As this graph shows, as the count of breaks on a particular pipe increases, the average duration until the next break decreases. Although this phenomenon has been observed by others (O'Day, et al., 1985; Ellison, et al., 2014), seldom has the significance of this effect been so clearly seen.

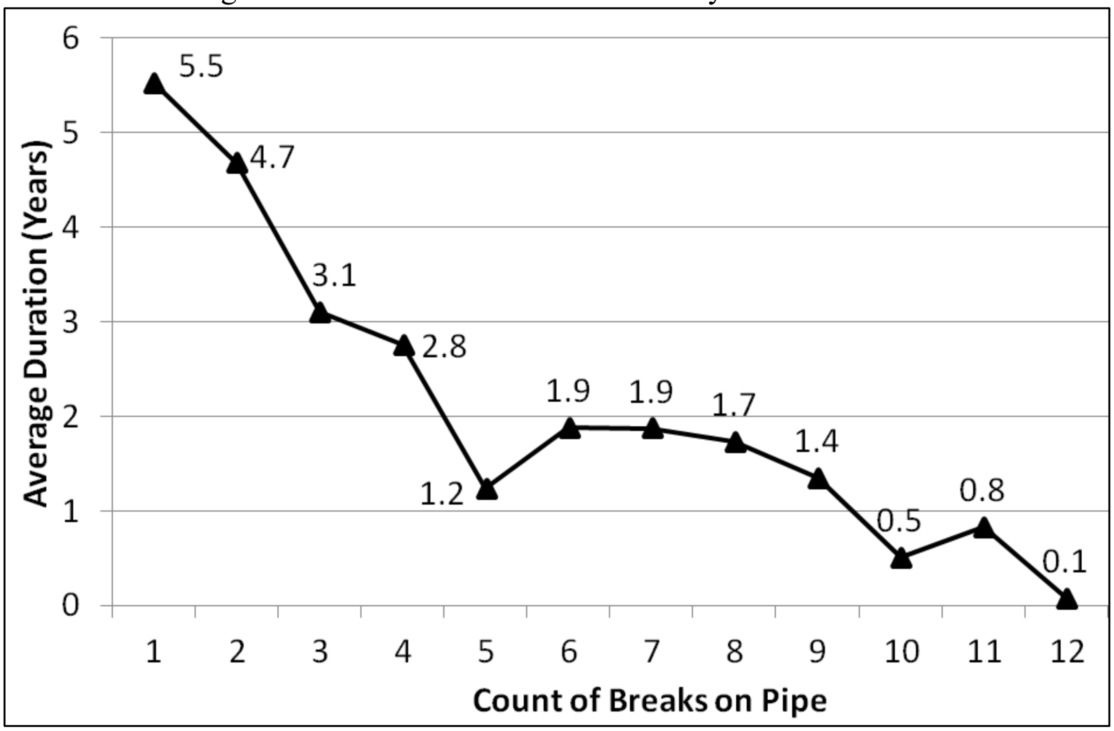


Figure 6. Data driven renewal investment projections confirm that EBMUD should increase near term renewal investments.

The predictability of the second, third, and subsequent breaks appears to be independent of the age and other factors that may have influenced the first break. For example, Figure 7 shows the same data with performance also summarized by pipe age. If age were a primary driver for predicting future breaks at a pipe level, we would expect older pipes to have a shorter duration between breaks than younger pipes. However, the data does not support this conclusion. Instead, the data supports the conclusion that regardless of whether a pipe is 30 years old or 60 years old, Type I or Type II, subjected to high pressure or low pressure, the intervals between breaks appear to be driven by the count of historic breaks rather than other factors.

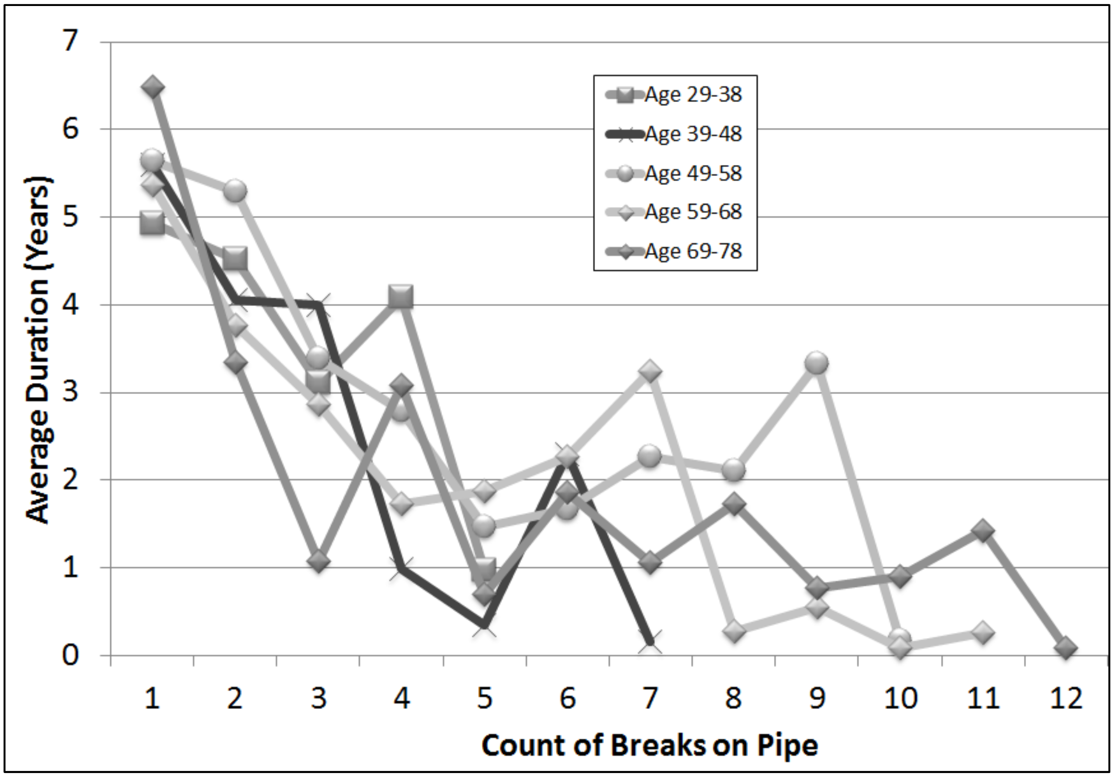


Figure 7. The duration until the next break is primarily driven by break count, not other factors such as age.

This finding was validated by a separate analysis to determine the effectiveness of various replacement scenarios. Effectiveness was measured based how many breaks would have been avoided per mile replaced if certain segments of the pipe population were replaced on January 1, 2008. Table 1 summarizes the results of this analysis and shows that replacement of pipes that have broken at least three times would have been a more effective than other replacement strategies.

Table 1. Effectiveness of various replacement scenarios

Scenario Title	Breaks Since 2008	Miles	Breaks Avoided per Mile Replaced
Random	734	1122	0.65
Steep Ground	422	591	0.71
East	386	441	0.88
East & LEP	378	416	0.91
High Pressure	132	114	1.16
East, LEP, Old, Steep, High Pressure	25	17	1.50
Type 1	47	20	2.40
Small Diameter	57	22	2.54
3 Breaks prior to 2008	63	24.3	2.60

WHICH CONDITION ASSESSMENT TESTS ARE MOST USEFUL IN FOCUSING RENEWAL INVESTMENTS?

Table 2 summarizes the number of samples available and analyzed by test. The primary focus of the analysis was to determine which physical tests, if any, are good indicators of future breaks. Two analysis methodologies were employed:

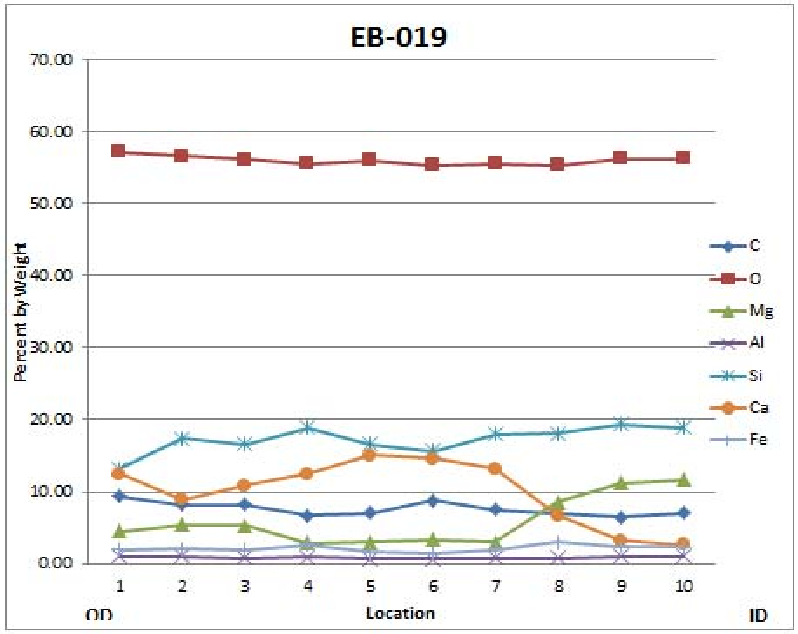
- Comparative Tests - Where multiple sources of test data exist on a single sample, test results were compared to determine whether they would indicate similar pipe condition. In general, if industry accepted tests correlate well, this would increase overall confidence in the usefulness of the data. If tests do not correlate well, this indicates that one or both tests are not accurately reflecting the condition of the pipe sampled.
- Anticipated verses Measured Condition - This analysis uses break history to measure the expected state of a pipe (i.e., the Anticipated Condition). If the Measured Condition from physical test results correlates with the Anticipated Condition, that information would support the conclusion that the test is a reliable measure of condition.

Table 2 – Summary of Test Data Analyzed

Test	Count of Samples
SEM Test	61
Stain Test	70
Coupon	24
Full Circumference	46
Diameter to Thickness Ratio	70
Flexural Strength Up	16
Flexural Strength Down	16
Crush Strength	44
Tensile Strength	15
Echologics	52

Analysis results suggest that Scanning Electron Microscopy with X-ray microanalysis (SEM) testing and inner wall Stain testing are the most accurate in predicting future breaks. The simplest test, Stain testing, has several limitations. Results can vary considerably around the circumference of the pipe, casting doubt on the usefulness of a single measurement. For this reason, the extraction of full-ring pipe samples is preferred, even though this method can be more expensive than extracting a coupon.

While 94% of Stain and SEM results correlated on the inner wall, only 20% correlated on the outer wall. For example, Figure 8 shows a sample with a strong correlation between SEM and stain test on the inner wall but a poor correlation on the outer wall. This quantifies the most significant problem with Stain testing which is that it does not distinguish between simple carbonation and calcium loss. Carbonation without calcium loss was frequently found on pipe exteriors, and carbonation alone should have minimal effect on pipe integrity. For this reason, SEM tests were found to be generally more useful (they are not fooled by carbonation), but these tests cannot be performed in the field and are more costly. Note, as a possible solution to this, EBMUD purchased a hand-held X-ray fluorescence (XRF) tool for field use but it was not available at the time of this study and therefore the results were not validated.



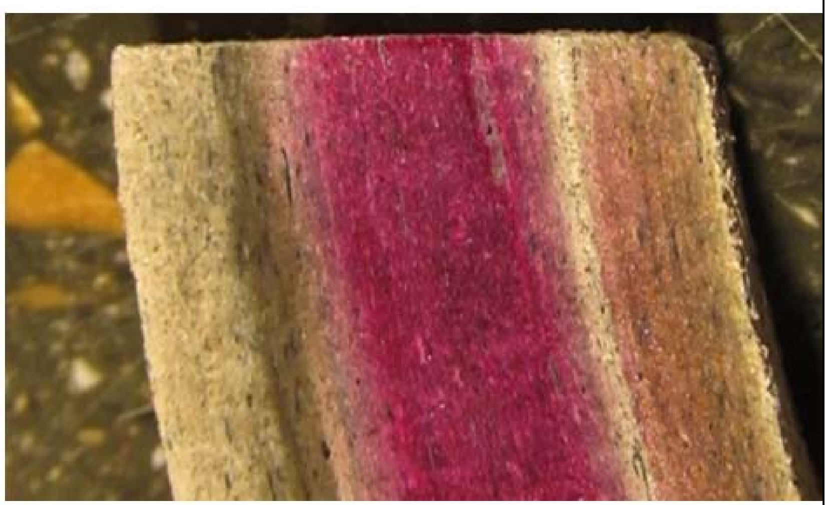


Figure 8. The primarly limitation of Stain testing is that it cannot be trusted to measure deterioration on the outer wall as shown in this sample where calcium content is high but significant stain loss is shown on the outter wall.

This study recommends collecting one of these two sources of data during AC break response. The value this data will provide to future renewal decision makers will far outweigh the minor cost of performing these tests on a pipe that has already been exposed. SEM is appropriate for utilities with large quantities of AC pipe, utilities with access to cost effective SEM testing, and where external pipe wall deterioration is prevalent. Stain testing is more appropriate when utilities have moderate amount of AC pipe, limited access to cost effective SEM testing, and where external pipe wall deterioration is not prevalent. In either case, tested protocols should be established, samples should be photographed, and test results should be dated and located (ideally using a GPS location).

As condition assessment data becomes available, it should be incorporated into guidelines for selecting and prioritizing pipes for renewal as well as protocols for establishing renewal project extents. For example, if tests results show significant deterioration in pipes close to a failed pipe, project limits should be expanded to include such deteriorated pipes. If test results show minimal degradation in pipes close to the failed pipe, project limits should not be expanded.

EBMUD and other utilities are cautioned to be judicious in replacing pipes that don't represent a clear risk. Although this study has shown that pipes with certain characteristics (e.g., Type I) have greater propensity to break, 91 percent of the system by count and 80 percent of the system by length have not had a recorded break since 1990 (when recording of breaks in a computer data base began). Evidence suggests that most of these pipes have many additional years of potential service. If a pipe is replaced before it fails, its remaining usefulness is forever wasted. In addition to these likelihood factors, consequence factors are also to be considered, including property damage factors, community impact factors, environmental impacts, and system disruption factors.

EBMUD engaged a testing company for acoustic velocity testing of 52 AC pipes. Even with some known limitations in measuring the effectiveness of acoustic velocity testing, results were promising. If the technology can improve to account for PVC repairs and be validated by more robust testing, this technology could prove to be a cost effective condition assessment approach for AC pipe.

No predictive value was found in the mechanical strength tests (crush, tensile tests, and bending) that were performed. With fair consistency, test strengths exceeded original specifications even on samples extracted from break repair sites. A possible explanation is that the degree of degradation is more important than actual strength of the material, since degradation (loss of effective wall thickness) affects not just strength but the bending modulus. Loss of effective wall thickness also amplifies the effects of material defects. For example, a material defect that is buried within a 0.5 inches thick pipe wall becomes more salient when the material degrades to an effective thickness of 0.25 inches.

CONCLUSIONS

The study team recommends the following for an effective AC pipe management strategy at EBMUD:

1. Within the next 5 years, increase the rate of AC pipe renewal from its current level of approximately 2 miles per year to approximately 5 miles per year.
2. Plan to further increase AC pipe renewal, targeting approximately 10 miles per year by 2030.

3. Re-evaluate the renewal rate by repeating the analyses every 5 years.
4. Select pipe for renewal based primarily on the historical number of break repairs. Use other factors (pipe diameter, type, soil, pressure, and condition) to select pipe for judicious project extensions.
5. Collect condition test data during break repairs and at other low-cost opportunities, and record these data in the GIS. Data may come from phenolphthalein stain testing or more sophisticated methods (SEM/EDS or XRF).
6. Consider the use of *structural* rehabilitation as an alternative to open-trench water main replacement. Before adopting a method, verify its ability to withstand fracturing of the host pipe, while pressurized.

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Pipe Bursting Asbestos Cement Pipe: The Process Is Established but What's Next

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Abstract

The City of Casselberry completed its $10.3 million asbestos cement (AC) pre-chlorinated potable water main pipe bursting project in April of 2014 which replaced approximately 35 miles of AC pipe. City staff has worked closely with the contractor, engineers and regulators from local and federal government to fully understand the applicability of the National Emissions Standards for Hazardous Air Pollutants (NESHAP) to pipe bursting of asbestos cement pipe. The Environmental Protection Agency and industry representatives have recognized the need to understand the potential environmental impacts of AC pipe rehabilitation and have tasked the Water Research Foundation and the Battelle Institute with studying the various methods of AC pipe rehabilitation. The environmental impacts of pipe bursting AC pipe have been analyzed with the Casselberry Water Quality Improvement Project as its pilot project. Results of the WRF study indicate that bursting AC pipe is more environmentally friendly than removing the existing AC pipe while providing the option to rehabilitate the existing pipeline in place. This paper will present the results of WRF Project #4465 while clearly describing how to burst AC pipelines and meet all existing regulations. This paper describes the challenges and successes of implementing a pipe bursting project, from field application of pipe bursting technology to working directly with regulators and right-of-way controllers who may be skeptical about pipe bursting AC pipe. A potential path forward through submission of a potential Administrator Approved Alternate to EPA that accepts a streamlined AC pipe bursting process is also presented.

INTRODUCTION

The City of Casselberry started its major asbestos cement (AC) pipe bursting project in 2009 in response to the American Recovery and Reinvestment Act's call for *Shovel Ready* projects. The project started as a $3 million project and grew to $10.3 million as the success of the project continued. The city of Casselberry, its contractor Killebrew, Inc., and construction inspection engineer, CPH Engineers, Inc.

worked very closely with regulators from the local and federal governments as well scientific agencies, such as the Water Research Foundation and Battelle Memorial Institute, to fully understand the applicability of the National Emissions Standards for Hazardous Air Pollutants (NESHAP) to pipe bursting of asbestos cement pipe. Understanding how the Clean Air Act that was written in the early 1970's applied to pipe bursting was not an easy task. Many regulators and other people not familiar with pipe bursting envisioned the airborne release of asbestos particles during the pipe bursting process. This is simply not what happens while the pipe bursting work is occurring and the project team worked diligently to dispel the myths.

The project team understood the importance of successfully implementing what would become the largest AC pipe bursting project in North America and working closely with all regulatory agencies to meet every aspect of regulations that controlled the work. The project was federally funded and would be required to stand up to scrutiny through a comprehensive audit at the close of the project. The project could also serve as a guideline to other projects that could build on the progress made by the project team fully understanding the complicated regulations and applying them to pipe bursting of AC pipe. The project team consistently volunteered the project for scientific study and analysis and routinely spoke about the project. Environmental Protection Agency officials have recognized the need for additional research into the environmental impacts of AC pipe rehabilitation methods. They tasked the Water Research Foundation and the Battelle Memorial Institute through WRF Project #4465 to analyze the available methods of AC pipe rehabilitation and their environmental impacts. The project team quickly volunteered the Casselberry project as a pilot project for the Battelle Memorial Institute's study.

The Battelle Memorial Institute in conjunction with the project team, planned a week of on site field research to witness rehabilitation of a 775-ft section of AC pipe and to collect air, soil and water samples during the process. The Battelle Memorial Institute followed key EPA sampling guidelines, such as ISO Method 10312, EPA Method 600/R-93/116 and EPA Method 100.2, during the pilot study sampling activities. Air sampling limits for asbestos fibers came back well under the established Occupational Safety and Health Administration's (OSHA) established limits for permissible asbestos fiber limits. The result in soil sampling pre and post levels show almost no change in presence of asbestos fibers in the soil after pipe bursting. Post pipe bursting water samples showed no levels of asbestos fibers that exceeded EPA maximum contaminant levels (MCL) in the water although one pre pipe bursting sample exceeded the EPA MCL but the sample appeared to be faulty. In general, the Battelle Memorial Institute's work summarized that there is no evidence to support that the bursting of AC pipe has any negative impacts on the environment or the workers performing the work.

PROJECT HISTORY

The City of Casselberry is a medium size town in sub-urban Orlando that is considered to be 95% developed. Much of the development occurred between 1950 and 1980. This time frame occurs with the increased popularity of installing AC water mains within the United States. There are widely varying estimates as to the amount of AC pipe installed within the United States and Canada but some estimates conclude there could be as much as 630,000 miles installed (Von Aspern, 2009). Almost 50% of the potable water distribution network within the City of Casselberry was AC pipe prior to the start of the Water Quality Improvement Project. The majority of this pipe is smaller diameter AC pipe (under 12”) that displays higher rates of failure than the larger diameter AC pipe (AWWA, 2012.)

Prior to 2009, the City was appropriating $300,000 per year to replace existing potable water mains throughout the City. The City owns and maintains 215 miles of potable water main in its distribution network. The $300,000 previously appropriated replaced approximately one mile per year and this replacement schedule would require 215 years to replace the potable water distribution network. The anticipated fifty year service life of the existing asbestos cement pipe was almost over as the pipe was already forty years old and the current replacement schedule was not sustainable (Ambler, et. Al, 2014). Funding for replacing the AC pipe did not generate a new source of revenue for the city of Casselberry, which further complicated replacement of the existing AC pipe. Luckily, the City applied for and received grant and loan funding through the Florida Department of Environmental Protection (FDEP) State Revolving Loan Fund (SRF) program and the American Recovery and Reinvestment Act (ARRA) to support the project. In development of the project, the City identified the locations of AC pipe within their network that suffered significant pipe failures and were nearing the end of their predicted service life and the City then designed the comprehensive Casselberry Water Quality Improvement Projects.

The Casselberry Water Quality Improvement Projects was a pipe bursting project that did not require a permit from FDEP and typically no right-of-way acquisition, which made it *Shovel Ready*. To date, the project has received a total of $10.3 million in construction, engineering and administrative costs, of which $6.55 million was considered as principle forgiveness, or grant money (Ambler, et. Al, 2014.)

City staff utilized the City’s extensive geographical information system (GIS) files to identify the distribution pipes that were nearing the end of their service life. City staff also compared these areas with historical failure rates to prioritize pipe replacement areas. The City selected pipe bursting as the most rapid and effective trenchless technology pipe rehabilitation method with the least environmental and social impacts. The City also realized significant economic benefits by minimizing construction schedule, resident/customer impacts and environmental impacts. Unfortunately, pipe bursting of AC pipe has not been widely accepted throughout the United States. This is primarily due to existing regulations that do not accommodate technological development, dramatic variation of the application of these regulations and ignorance and fear of the actual hazards of asbestos (Ambler, et. Al, 2014.)

NESHAP SYNOPSIS AND HOW TO MEET REGULATIONS WHILE BURSTING PIPE

Much of the confusion surrounding regulatory control of pipe bursting of AC pipe is the pipe bursting work is not addressed by the Drinking Water Act (DWA) but rather the Clean Air Act (CAA). Many people would not correlate the CAA with governing rehabilitation work on a buried pipeline. However, EPA has determined that demolition of the existing AC pipe during the process of pipe bursting triggers the National Emissions Standards for Hazardous Air Pollutants (NESHAP). NESHAP is a sub section of the CAA that is aimed at controlling release of hazardous industrial chemicals into the air or work environments. Asbestos was one of the first industrial chemicals as regulated by NESHAP. Asbestos was considered to be a “magic” mineral during the first part of the 20th century due to its flexible, non-destructible and heat resistant nature. This perception changed dramatically as the adverse health effects of occupational asbestos exposure started being known (Ambler, et. Al, 2014).

Picture 1. Fractured AC pipe resulting from pipe bursting as it will remain in the ground.

EPA defines two categories of non-friable asbestos containing material (ACM), Category I and Category II non-friable ACM. Category I non-friable ACM is any asbestos-containing packing, gasket, resilient floor covering or asphalt roofing product that contains more than 1% asbestos as determined using polarized light microscopy (PLM) according to the method specified in Appendix A, Subpart F, 40 CFR Part 763 (Sec. 61.141). Category II non-friable ACM is any material, excluding Category I non-friable ACM, containing more than 1% asbestos as determined using PLM according to the methods specified in Appendix A, Subpart F, 40 CFR Part 763 that, when dry, cannot be crumbled, pulverized, or reduced to powder by hand pressure (Sec. 61.141) (Ambler, et. Al, 2012.)

In 1990, EPA issued clarification that AC pipe that has undergone pipe bursting is considered regulated asbestos containing material (RACM) and is governed by NESHAP. RACM is directly defined as friable asbestos material or non-friable ACM that will be or has been subjected to sanding, grinding, cutting, or abrading or has crumbled, pulverized or reduced to powder in the course of demolition or renovation operations (www.epa.gov). It is arguable that AC pipe that has undergone the pipe bursting process cannot be further crushed by hand to release asbestos fibers (Ambler, et. Al, 2014).

Many engineers, contractors and utility providers strongly disagree that pipe bursting AC pipe converts the previously non-RACM AC pipe into friable RACM. EPA maintains that pipe bursting AC pipe does convert the AC pipe into friable

RACM. However, a working procedure has been developed in Florida that regulators and industry members (Municipalities, engineers, and contractors) are utilizing. This procedure complies with each element of NESHAP (40 CFR part 61, subpart M (61.140-61.157)) and is described below (Ambler, et. Al, 2012.):

- File a Notice to EPA or Its Designee (61.145(b)). NESHAP specifies salient information that must be included on the notice. FDEP has an available form 62-257.900(1) that requires this information. The form is a single page form that has to be signed only by the utility owner.
- Provide for Emission Control during Renovation and Disposal
  There can be no visible emissions from the work [pipe bursting] per 61.150(a). With pipe bursting, this can be accomplished because the AC pipe is wetted within any excavation, and non-power saw tools are used to cut the pipe (chain cutter, handsaw).
- Comply with Inactive / Active Waste Disposal Site Requirements (61.151 / 61.154). NESHAP provides for disposing of RACM on the site of the demolition/renovation work or at a waste disposal site.

Currently regulators interpret NESHAP such that the work site is considered a waste disposal site for pipe bursting projects. Numerous options are provided in NESHAP to prevent asbestos exposure. These options include: no visible emissions from the site, fencing and posting signs around the site, have a natural barrier (cliffs, lakes or other large bodies of water, deep and wide ravines, and mountains) around the site, or cover the RACM with two feet of compacted non-asbestos containing material. With pipe bursting, the two feet of cover is virtually always provided because most all buried AC pipeline maintain greater than 2' depth of cover (Ambler, et. Al, 2012).

- Comply with Inactive Waste Disposal Site Deed Notation and Alternative (61.151(e))
  NESHAP requires that a notation to the deed of a facility property be recorded within sixty days of a waste disposal site becoming inactive. A site is deemed inactive when disposal of RACM is completed. Applying this to pipe bursting projects, a site is deemed inactive when the project is completed. The notation is to contain the following information (Ambler, et. Al, 2012):

    1. The land has been used for the disposal of asbestos-containing waste material;
    2. The survey plot and record of the location and quantity of asbestos-containing waste disposed of within the disposal site required in Sec. 61.154(f) have been filed with the Administrator; and
    3. The site is subject to 40 CFR part 61, subpart M (Ambler, et. Al, 2012.)

Most of the buried AC pipeline infrastructure owned by the majority of utility providers within the United States lies within public right-of-ways. However, public right-of-ways do not maintain a property deed where the restrictions NESHAP references can be directly met. This conflict brought many industry members and the contractor for the Casselberry Water Quality Improvement projects to Washington D.C. to meet with top EPA staff to discuss pipe bursting and the applicability of NESHAP to pipe burst AC pipe. EPA officials embraced the environmental, social and economic benefits of pipe bursting AC pipe and understood the risks of asbestos exposure due to pipe bursting AC pipe would be mitigated over traditional pipe removal methods. While pipe bursting was met with a positive response, modification of the existing NESHAP regulations would require an Act of Congress to complete. EPA officials recommended industry representatives present the EPA Administrator with an "Administrator Approved Alternate" process that can cover AC pipe bursting. To date, there has never been an "Administrator Approved Alternate" process approved to supersede NESHAP nor has any guidance been given to prepare the Administrator Approved Alternate. Industry representatives are currently working through the Administrator Approved Alternate Task Force to develop a suitable document to submit to EPA (Ambler, et al., 2012).

EPA'S STUDY OF ENVIRONMENTAL IMPACT OF ASBESTOS CEMENT (AC) PIPE RENEWAL TECHNOLOGIES

The Water Research Foundation (WaterRF) and EPA Office of Research Development (ORD) recently funded a study of the environmental impact of various AC pipe renewal technologies, including pipe bursting among others. The results of the study are set to be published in the fall of 2015 via a WaterRF project report and most likely a peer-reviewed journal article, which will be valuable when preparing the Administrator Approved Alternate. At the time of this paper's publication, one AC pipe bursting demonstration had been completed with air, water, and soil samples being collected. The water and soil samples were collected prior to the demonstration and post-pipe busting samples will be collected for comparison to determine the impacts of the project on water quality and soil contamination. Initial results show no adverse impacts to either the soil or wate. (Ambler, et. Al, 2014).

As part of Phase 2 (i.e., Technology Demonstration and Evaluation) of Water Research Foundation (WaterRF) Project No. 4465, Environmental Impact of Asbestos Cement (AC) Pipe Renewal Technologies, the City of Casselberry was identified as one of the only municipalities in the United States actively performing pipe bursting on AC pipe. For this reason the City of Casselberry was selected as a site where the technology of pipe bursting could be adequately demonstrated and its impacts on the environment could be properly evaluated.

In the summer of 2013, Battelle was onsite in Casselberry, FL to observe the renewal of a 775-ft section of AC pipe (ca. 1972) and to collect air, soil, and water samples during of the process. Over the course of a week, five (5) bursting runs ranging from 125 to 190-ft in length were performed to replace 450-ft of 8-in and 325-ft of 12-in AC pipe. The AC pipe was replaced with 12-in high-density polyethylene (HDPE) pipe.

To determine the impact to the environmental as a result of pipe bursting AC pipe, air, soil, and water samples were collected while onsite. Six (6) air samples were collected during all major activities using two SKC AirChek® XR5000 personal air sampling pumps with approximate flow rates of two (2) liters per min (LPM). Six (6) soil samples were collected from the side walls of access pits following excavation of the pit but prior to any pipe related activities. Six (6) post-renewal soil samples were collected from the same pit wall locations months after the completion of the renewal work and compared to the pre-renewal soil samples. A total of four (4) water samples were collected – two (2) pre-renewal and two (2) post-renewal – from a residential water service line and fire hydrant. A summary of the sampling results is presented in Table 1. Note that all samples were only analyzed for asbestos and no other contaminates.

The asbestos concentration of each air sample (see Table 1) is below the analytical sensitivity. The analytical sensitivity of each sample is below the 8-hr time-weighted average (TWA) permissible exposure limit (PEL) of 0.1 s/cc set by the Occupational Health and Safety Administration (OSHA). This indicates the workers were not exposed to dangerous levels of airborne asbestos throughout the duration of the project.

Table 1. Summary of Asbestos Sampling Results for Air, Soil, and Water

Sample Type	No. of Samples	Analytical Sensitivity Range	Sample Result Range	Analytical Method
Air	6	0.0036 - 0.0042 s/cc	BAS	ISO Method 10312
Soil (Pre-renewal)	6	NA	ND - Trace (<0.25% visual estimate)	EPA Method 600/R-93/116
Soil (Post-renewal)	6	NA	ND - Trace (<0.25% visual estimate)	
Water (Pre-renewal)	2	0.17 - 0.35 million structure/L	0.87 - 20.07 million structure/L	EPA Method 100.2
Water (Post-renewal)	2	0.08 - 0.09 million structure/L	0.09 - 0.94 million structure/L	

s/cc = structure per cubic centimeter (mL)
BAS = below analytical sensitivity
NA = not applicable

The results from the pre- and post-renewal soil samples (see Table 1) show essentially no change in asbestos levels within the soil. Although some locations saw an increase of asbestos by trace amounts, other locations saw a decrease in asbestos concentration by trace amounts or saw no change at all. With no significant change in the asbestos concentration between the pre- and post-renewal samples, there is no evidence of upward migration of the asbestos fibers within the soil column.

Water sample results for the pre-renewal samples show one (1) sample with an asbestos concentration of approximately 20 million structures/L, which is almost three times the USEPA maximum contaminate level (MCL) for asbestos in drinking water (i.e., 7 million structure/L). The sample was collected from a fire hydrant prior to any pipe related activities and is believed to have been inadequately flushed prior to collection. The post-renewal water samples show a dramatic decrease in asbestos concentration, especially the sample from the hydrant, which saw a reduction in asbestos of nearly 90%. Both post-renewal samples were below the EPA MCL, therefore, posing no health risk to consumers. Note that the new HDPE line is still connected to AC lines at three locations and the presence of asbestos in the drinking water is likely to continue, albeit at lower concentrations than before.

Based upon the results from the air, soil, and water samples collected from the Casselberry site there is no evidence to support that the bursting of AC pipe has any negative impacts on the environment or the workers performing the work.

FIELD OBSERVATIONS

The Casselberry Water Quality Improvement Projects lasted well over four years and installed almost 35 miles of HDPE through pipe bursting. Key construction engineering inspection field staff executing the day to day operations has made significant key observations. The original project documents as bid required a Bursting Plan be submitted prior to mobilizing to the new project area and starting bursting operations. A Bursting Plan is a modification of the original plan sheets. Similar to the original plans, a Bursting Plan should be based on the GIS information supplied by the owner or client of the project, available survey information, as-built information and/or field verified information. These plans should depict all entrance and exit pits, service connection pits, fire hydrants, blow-off connections and any other miscellaneous appurtenances that are proposed to be replaced or added. Each section of pipe or Burst Section should be labeled with the approximate length, size of existing pipe, size of proposed pipe that will be used to replace the existing pipe and the associated pipe materials. The plan should indicate all existing isolation points such as valves and dead end lines and any existing infrastructure that may have been installed on the system, such as line stop sleeves, abandoned valves, fittings, repair clamps, concrete restraints, etc. Other important information that should be noted on the Bursting Plan should be the approved pipe bursting procedures for the project, all of the standard project information such as general project area locations, street names, etc.

A Bursting Plan is useful information that can be used to satisfy the regulatory requirements of NESHAP previously outlined. However, the Bursting Plan is critical for the contractor when estimating how much preparation is required within the pipe replacement project area prior to starting work within the area. The Bursting Plan allows the contractor to layout the project area with the appropriate number of burst segments with appropriate burst lengths in order to accommodate for all known isolation points, utility crossings, naturally and mechanical limitations. There are limitations as to how much pipe a work crew can reasonably install in a work day and these limitations should resonate throughout the Bursting Plan. The Bursting Plan also informs the contractor of what existing infrastructure needs to be located and tested prior to commencement of any of the pipe replacement activities. If the existing distribution system does not have enough isolation valves to meet maximum water outage limits required for the project, the contractor must provide for temporary components such as line stops, valves, services.

A Bursting Plan is used by the contractor, engineers and owner in developing a bursting schedule and tracking submittals. The bursting schedule can then be used to coordinate fusion of each of the burst sections. The replacement HDPE pipe can be staged in a long linear staging area and fused in sections to make one longer section of pipe that will be pulled into place for each Burst Section. After the final pipe is fused, hydrostatic pressure testing and bacteriological sampling can be performed on the final pipe. The bursting schedule helps minimize redundant bacteriological sampling for samples that have short expiration requirements. A 30 day expiration schedule for the bacteriological sample regulates how long a fused section remains on the staging area before the pipe is installed. These three steps are part of the pre-chlorinated potable water main pipe bursting process approved by the Florida Department of Environmental Protection (FDEP). FDEP considers this work to be rehabilitation of the existing pipeline and allows the pre-chlorinated potable water main pipe bursting work to occur without a permit for up to two pipe sizes larger than the existing pipe. Proper management of fusing, hydrostatic pressure testing and bacteriological sampling can result in direct cost savings to the contractor. A well-developed Bursting Plan is not only critical to the organization and coordination of the construction activities but critical to helping the project owner stay in compliance with the governing agencies and minimizing the costs of the project.

MOVING FORWARD

It's been over four years since industry representatives met with Washington, DC EPA staff to discuss the applicability of NESHAP to pipe bursting AC pipelines and work towards developing a reasonable and practical solution to accommodating new technological developments, such as pipe bursting, within the existing NESHAP framework. EPA staff had acknowledged the potential difficulty in applying NESHAP Deed Notation requirements to AC pipe bursting within public rights-of-way. During the meeting with EPA, a video of several physical demonstrations of AC pipe bursting were shown that clearly indicated the minimal environmental impacts of pipe bursting and dispelled myths that AC pipe bursting released an explosion of asbestos fibers into the air. It is possible that AC pipe bursting has been given a bad reputation specifically because of the misconceptions of AC pipe bursting. EPA staff in attendance of the meeting with industry representatives expressed a positive attitude towards pipe bursting of AC pipe after being presented with video demonstrations of the process. EPA staff suggested industry representatives submit an “Administrator Approved Alternate” for the EPA Administrator considers as an alternate process to existing NESHAP regulations.

An Administrator Approved Alternate is intended to allow the EPA Administrator and staff to approve alternate technology or practices without having to modify NESHAP, which is federally codified. Industry members that have been following the pipe bursting of AC pipe issue are pleased with the opportunity to pursue an Administrator Approved Alternate and are working toward this objective. However, at this time, there does not appear to be any guidance documents or previous examples of an EPA Administrator Approved Alternate to reference. An Administrator Approved Alternate has not been developed for any technology or practice to date. An AC Pipe Bursting Task Force has been assembled to develop this document. (Ambler, et. Al, 2012.)

The Administrator Approved Alternate and it is intended to provide procedures for working with buried AC pipelines. The exemptions and clarifications listed early will be included so that one, comprehensive document, specific to buried AC pipelines, will be available for use nationwide and that any type of work on buried AC pipelines will be uniformly practiced and regulated, regardless of which State the work may be located in. (Ambler, et. Al, 2012.)

Collaborative efforts among industry members have been on-going since November 2010 to draft the Administrator Approved Alternate. Once the first draft is prepared, it will be submitted to EPA’s Washington, DC office for review and consideration. In the meantime, to satisfy the deed notation requirement, a notice is being recorded to public records that contain all required information for ongoing projects in the State of Florida. (Ambler, et. Al, 2012.)

EPA's ORD has set a goal to generate the science and engineering needed to improve and evaluate promising innovative technologies and techniques that will reduce the cost and improve the effectiveness of operation, maintenance, and replacement of aging and failing drinking water and wastewater treatment and conveyance systems. Existing technologies need to be applied in unconventional ways. Emerging technologies and innovative thinking will be at the forefront of creating powerful, secure, cost-effective, and reliable water infrastructure (EPA Addressing the Challenge through Science and Innovation, 2010). Industry believes application of pipe bursting for AC pipe is a prime example of an emerging technology that should be approved and utilized to mitigate the accelerating costs of AC pipe replacement. (Ambler, et. Al, 2012.)

CONCLUSION

Scientific research and testing of direct field implementation of asbestos cement pipe bursting by both utility owners and EPA commissioned scientists has clearly illustrated the asbestos cement pipe bursting is a safe and environmentally friendly method for rehabilitation asbestos cement pipe. The City of Casselberry, in conjunction with its contractor, Killebrew, Inc. has performed Negative Exposure Assessments on pipe bursting work confirming no asbestos fibers are released during rehabilitation activities above established OSHA limits for asbestos work. Water Research Foundation Project #4465 has come to the conclusion that “there is no evidence to support that the bursting of AC pipe has any negative impacts on the environment or the workers performing the work.” A safe, simple method for executing an asbestos cement pipe bursting project while meeting all existing regulations has been established by industry and the City of Casselberry. This safe, simple method for asbestos cement pipe bursting has been validated by scientists hired by EPA. Supporters of AC pipe bursting believe there should be no hesitation by owners of asbestos cement pipe to move forward in rehabilitating their failing asbestos cement pipe via the pipe bursting method.

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Management of a Pipe of High Concern for Failure: Asbestos Cement Pipes

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Abstract

There are concerns that asbestos cement (AC) pipes have reached or are close to their design life. These pipes have been extensively used around the world, and estimates are that 15 to 18% of the pipe in the ground in the United States is AC pipe. The Water Research Foundation has engaged in a few recent studies regarding the management of AC pipe that have provided some surprising results. Supplementing this recent research work with a careful literature review and interviews with some key utilities managing large quantities of AC pipe, we will provide a brief overview of the state-of-the science on AC pipe management. While AC Pipe was found to be at or near failure in some case study utilities, in the majority of the case studies the pipe had many years of expected remaining life. For management of high consequence pipe We discuss the method developed to assess the expected remaining life of AC pipe for use on high consequence pipe, and discuss the use of breaks and leaks for management of low consequence pipe. The case is made that AC pipe should not be targeted for removal unless it is performing poorly or is assessed to have little remaining life.

BACKGROUND ON AC PIPE

Asbestos cement (AC) pipe is a mixture of asbestos fibers and Portland cement that was then rolled over a mandrel to form a pipe of certain diameter. AC pipe is estimated to constitute approximately 15-18% of the installed North American pipe based on the AWWA 2002 Distribution System Survey which had 337 utility respondents (AWWA 2005) while an earlier AWWA survey which included 998 responding utilities identified approximately 15% of the North American pipe being AC (AWWA 1996). While 15-18% of installed water pipe is a considerable amount, in fact water utilities have a widely variable amount of AC pipe that they are actually managing. Some utilities have no AC pipe to manage, while other utilities have a majority of their pipe being AC. For those utilities that have a significant amount of AC pipe, a number of issues are of importance in managing that pipe, in particular, how long might the pipe last and when should the pipe be scheduled for replacement?

This paper/presentation will primarily focus on smaller diameter AC pipe, that covered by AWWA Standard C400, pipe in the range of 4-inch diameter to 16-inch diameter. These size ranges are believed to constitute the majority of the AC pipe in the ground, although we do not have a good national estimate of the amount of larger diameter AC pipe (greater than 16-inch diameter) compared with smaller diameter AC pipe. The AWWA standards on AC pipe were withdrawn in November 2008 (AWWA 2008).

AC pipe manufacturing is typically broken into Type I and Type II pipe, with both processes including lime [$Ca(OH)_2$] within the pipe material. Type I pipe was first introduced, and typically consisted of asbestos fibers and Portland cement formed and cured under moist conditions and at atmospheric conditions, with variations in the exact process. Type I pipe is reported to have 15.5% of the total weight being free lime. Type II pipe was introduced in the US in the mid-1930s and typically consists of mixture of asbestos fibers, Portland cement and silica powder, cured using high pressure steam. The silica in the Type II pipe combined with the free lime and other elements in the cement to form a number of stable substances, leaving free lime less than 1% of the total weight. In either case the ends of the pipe were typically machined slightly for joints, the machined part of the pipe would have a very smooth finish, where typically the remainder of the pipe had a waffle pattern. It is reported that most 1940s and later pipe sold in North America is Type II pipe (Hu 2013) and that Type II pipe is less subject to degradation.

This paper is primarily based on work from the Water Research Foundation (WRF) addressing asbestos cement pipe, especially a recently completed project (#4093) titled “Long Term Performance of Asbestos Cement Pipe.” Since the mid-1980s the WRF has spent approximately 25% of its research funding on infrastructure and pipeline issues, including AC pipe. At this time the WRF has two ongoing AC pipe-specific projects, the recently completed AC-pipe project, and a number of other studies that have relevant information. The highlights of this research will be provided in this paper and some implications for management of AC pipe at water utilities will be explored.

DETERIORATION AND FAILURE OF AC PIPE

In the long run, all pipes will fail. For the purposes of this paper the “failure” of a pipe will be identified as that time in which the utility decides the pipe should be significantly renewed or replaced. Thus, “failure” is not the same as a break. It is expected that a given length of pipe, say 1,000 feet, will typically have a few breaks before the pipe is considered “failed.” However, although there are models to predict optimum replacement interval based on the amount of money spent replacing pipe versus the amount of money spent repairing a pipe, and even more complicated models that include the carbon footprint of the different pipe materials, a pipe has “failed” and needs replacement when a utility considers it to have failed. A utility can continue to repair a troublesome pipe and provide generally good service for as long as it chooses to, in essence it is simply shifting costs to operations and maintenance and away from capital investment. However, there typically is a point at which a pipe's characteristics have deteriorated, in some way, to the extent that upon repair the next failure of that pipe, in a short timeframe, is a near statistical certainty.

AC pipelines, like all others, fail in a number of different ways.

First of all, AC pipe is fairly brittle and is therefore prone to failure where there is differential soil settlement. Thus, AC pipe can be prone to failure where installation was poor, or where the general soil environment consists of soils with high shrink/swell characteristics, or where there have been seismic events. With regard to seismic events specifically, AC pipe has performed poorly in seismic events, and while this poor performance typically does not rise to the point of utilities specifically targeting the pipe for replacement in seismically active areas, it is another risk factor associated with the pipe, and may reduce the tolerance at the utility for maintaining that pipe all other factors being equal (Eidinger 2012).

Second, AC pipe can fail through loss of strength through lime leaching or sulfate attack. In lime leaching the lime is lost out of the pipe due to leaching to the conveyed water, or due to leaching to external water, or a combination of the two. The lime can also react with acids present in the conveyed water or in groundwater or nearby soils. AC pipe is also susceptible to sulfate attack, where calcium-sulfur compounds are formed in place of the original lime. These calcium-sulfur materials occupy more volume (123 to 224%) than the solids they replaced. This causes swelling and can lead to expansion and destruction of the pipe (Hu 2013).

THE GOOD NEWS: SURVEY RESULTS AND REMAINING AC PIPE LIFESPAN ASSESSMENTS

The WRF research project that forms the primary basis for this paper, included significant utility participation. Two surveys were conducted, one was a brief survey sent to 160 utilities strictly focused on better understanding the AC pipe inventory in the ground and related break rates. A comprehensive survey was sent to 20 utilities that were official project participants and generally managed larger inventories of AC pipe. The comprehensive survey was designed to collect data on the current usage and operating/management practices for AC pipes in North America. It included questions about pipe inventory, pipe working environments, current AC pipe management practices (condition assessment and renewal practices), as well as health and safety and waste disposal issues. Eight of the participating 20 utilities also provided AC pipe samples for remaining pipe lifespan analysis.

Survey results indicated that the participating utilities had a large percentage of AC pipe – 43.2% of their pipe (16,238 km of AC pipe, out of 37,626 km of all pipe). The year of installation of these pipes is provided in Figure 1. For these utilities there was little usage of AC pipe prior to World War II. However, the installation of AC pipes increased during the post-war construction boom and peaked in North America in the 1950s and 1960s. The increase was consistent with the significant population growth in this period. Another peak was observed in the late 1970s and the early 1980s, just before the gradual phase out of new AC pipe installations in the early 1980s. Therefore, AC pipes have two predominant age groups: one ranges from 40 to 45 years, and the other around 30 years of age. Very few AC pipes have served more than 60 years. None of the participating utilities reported having lined AC pipes in service (Hu 2013).

The predominant pipe sizes, in the distribution systems of the participating utilities, were 150 and 200 mm (6" and 8") diameter. More than half of the AC pipes were 150 mm in diameter and less. These pipes are more vulnerable to soil movement. All three classes of AC pipe (Class 100, 150, and 200) were used by utilities for water distribution. However, the pipe class information obtained was incomplete. Although the utilities knew that Class 150 was predominant and Class 100 was the rarest class in their systems, the exact percentages for each class were unknown. Although the type of AC pipe material is critical for understanding deterioration characteristics, the utilities did not have pipe type information recorded. Type II pipe was introduced in the mid-1930s, so most AC pipes still in use are likely to be Type II based on their age alone (Hu 2013).

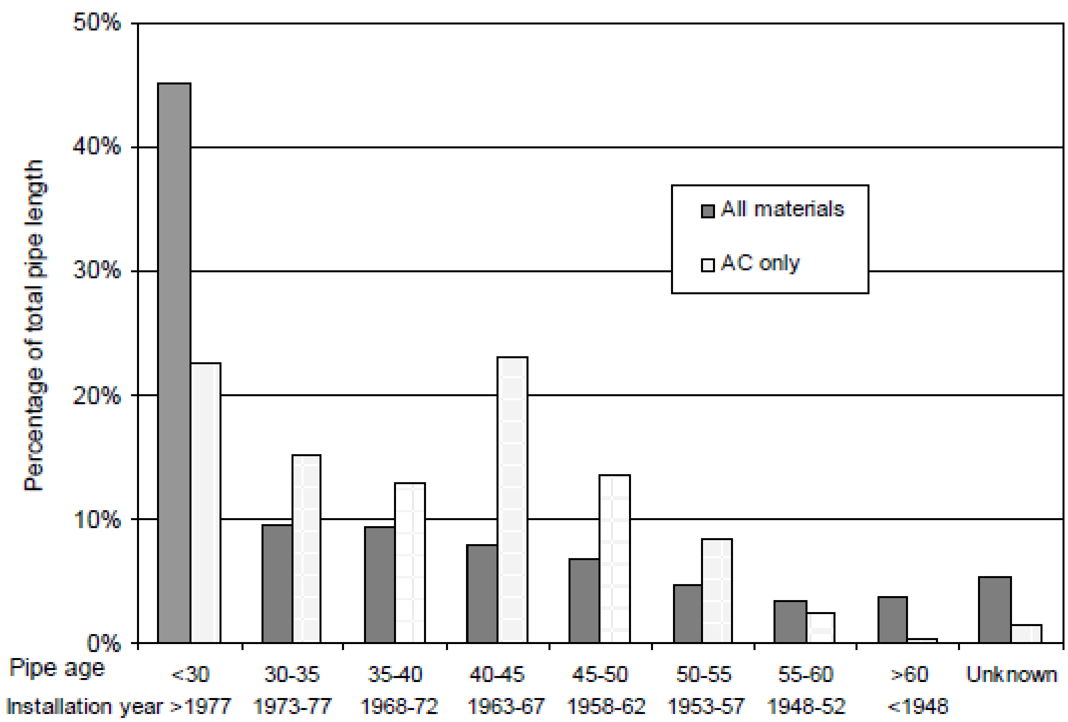

Figure 1: Installation Years for All Pipes and AC Pipes from Participating Utilities (Hu 2013)

The AC pipe break numbers were obtained for each participating utility for the six years from 2002 to 2007, inclusive. The data reflect substantial random variation in the annual breakage rate. The overall average breakage rate (all participating utilities and all 6 years reported) was 6.4 breaks/100 miles/year (4 breaks/100 km/year). The average breakage rate varied significantly from as low as 1.6 breaks/100 miles/year (1 break/100 km/year) to as high as 46.7 breaks/100 miles/year (29 breaks/100 km/year) (Hu 2013). A reasonable estimate of the average break rate in North America is approximately 25 breaks/100 miles/year for all pipes (Kirmeyer et al. 1994, AWWA 2005). The predominant failure mode for the AC pipe at the participating utilities was circumferential failure, which accounted for 70 percent of the AC pipe failures (Hu 2013).

For those eight participating utilities that provided pipe samples, their samples were analyzed and the data used to predict remaining service life of the AC pipe for those utilities. There was great variability in the results between utilities, some having AC pipe near its predicted service life, others having pipe with considerable service life left. There was also great variability in the data for a given utility in five of the eight utilities, with some pipe samples having much less remaining service life than other samples, but the other three utilities had much more consistent remaining service life predictions. Using median of the data for a given utility, six of the eight utilities had a median expected remaining service life of greater than 50 years of remaining life, some possibly as high as 170 years of remaining service life (Figure 2) (Hu 2013).

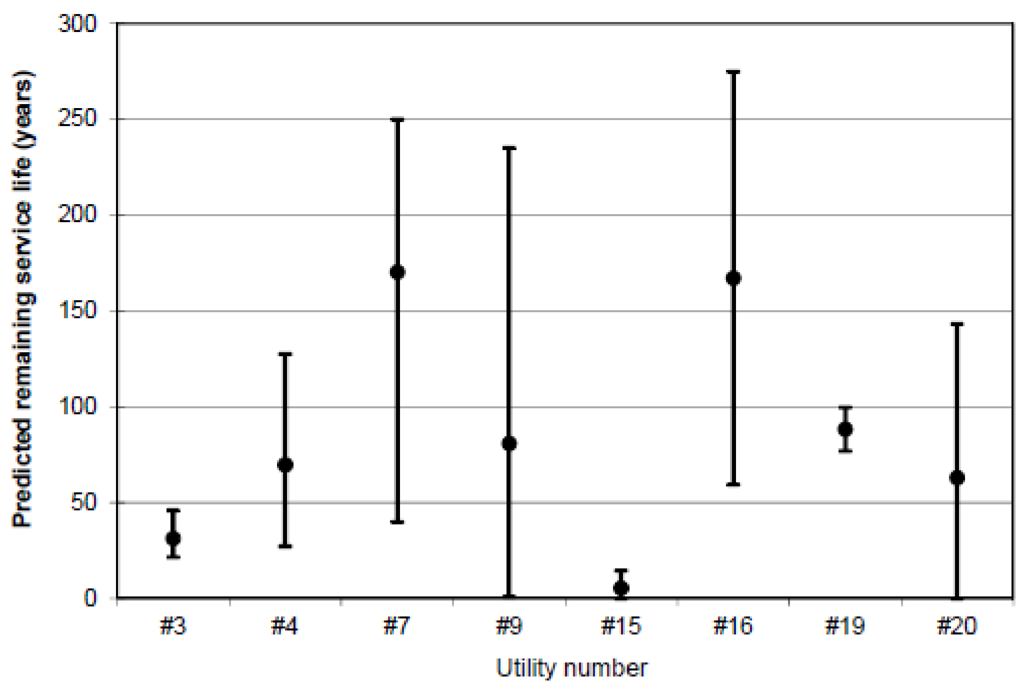

Figure 2: Predicted Remaining Service Life of AC Pipe for Eight Utilities with AC Pipe Samples and Complete Water Quality and Soil Data (Hu 2013)

ASSESSING THE REMAINING LIFE OF AC PIPE

The intent of predicting the remaining life of any pipe is to pre-empt unplanned and unacceptable failures, and to plan for an appropriate point at which the pipe should be renewed in some manner or replaced by a new pipe. With specific reference to AC pipe longevity assessments, the literature of predicting remaining life goes back to the 1960s, and various approaches have been used to evaluate the possible useful/remaining life of actual AC pipe.

Most approaches at assessing AC pipe life have addressed the loss of strength of the pipeline by assessing the loss of lime from the pipeline. Typically the loss of lime from an AC pipeline is assessed by taking samples of the pipeline, and conducting a phenolphthalein dye test on the pipe, frequently combined with crush tests on the pipe. The phenolphthalein dye test results in a pink stain associated with the pipe wherever there is still adequate lime as shown in Figure 3, which roughly correlate to that part of the pipe with remaining strength, the non-pink areas typically have little strength. These data on remaining wall thickness are correlated with results of crush tests to assess loss of pipe strength. As the amount of lime in the pipe is reduced, the area producing pink color will be similarly reduced, until lack of pink color can be roughly correlated with no residual strength in the pipe – meaning imminent collapse of the pipe – or multiple pipe breaks over a short distance (Figure 4). Based on analyses conducted in the United Kingdom, maximum observed rates of lime leaching in aggressive environments were 0.18 mm/year for internal deterioration and 0.27 mm/year for external deterioration (UKWIR 2005), the maximum noted deterioration rate in less aggressive conditions was 0.09 mm/yr (UKWIR 2005a). The key factors identified in the UK study that could be measured that would predict lime depletion of the pipe were identified as water alkalinity and pH, and soil pH (UKWIR 2005a).

Figure 3: Cross Section of Phenolphthalein-Stained Pipe (Hu 2013a)

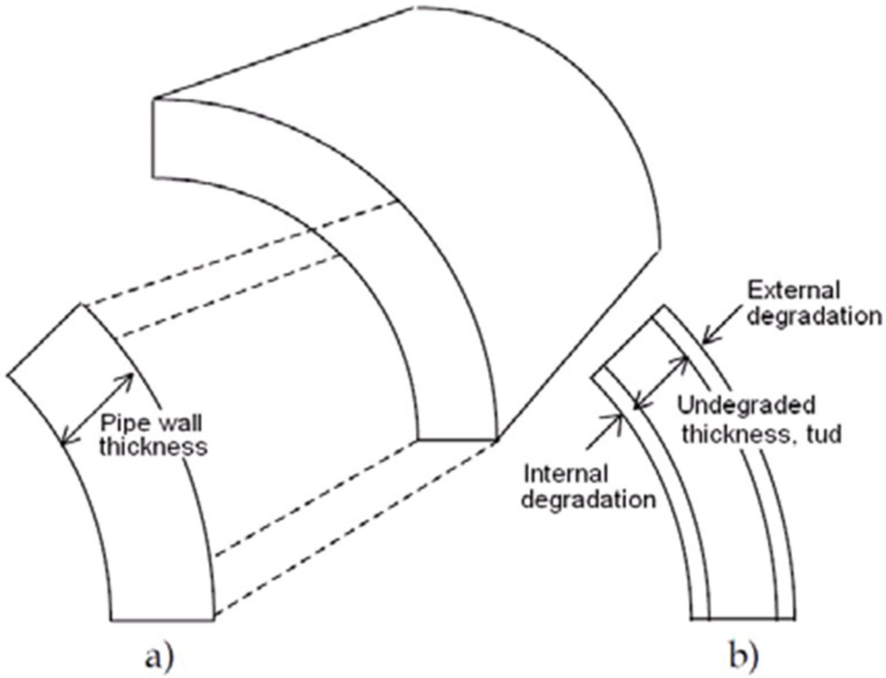

Figure 4: Pipe Wall Thickness and Assessment of Effective Pipe Wall Thickness (un-degraded pipe wall thickness) (Hu 2013a)

The WRF report *Guidance Manual for Managing Long Term Performance of Asbestos Cement Pipe* provides practical approaches to managing AC pipe in easy to follow steps. Use of this manual will allow utilities to assess their own AC pipe inventory, and to determine their rehabilitation and replacement needs, based on risk management and economic-based decisions. In this assessment the individual pipes to be managed should be placed into one of three inventory categories – high, medium, or low in terms of failure consequences. Much work is ongoing to better understand and define possible failure consequences, and thus to help in the identification of critical pipes (those with high consequences if they fail) but for the purposes of this assessment, and to help utilities get started with their assessment the Guidance Manual provides a starting point as given in Table 1 below. In this table a simplified approach is used where diameter of the pipe and traffic counts are used as a surrogate for criticality. A utility could use this as a starting point and scale up or down any particular factor to better suit their given situation. While sometimes difficult to assess a key consideration in increased losses associated with a break is longer shutdown timeframes. Thus, any high and also probably medium consequence pipes should be assessed relative to their shutdown and the location and operability of the valves necessary to accomplish shutdown (Gaewski 2007).

		Peak traffic count – vehicles per day		
		Important (≥ 1000)	Less-important (500 to 999)	Not-important (≤ 500)
Water pipe diameter or facility criticality	≥ 300 mm or critical facility	AC pipes with high consequence of failure	AC pipe with high consequence of failure	AC pipes with medium consequence of failure
	≤ 300 mm or less-critical facility	AC pipes with high consequence of failure	AC pipes with medium consequence of failure	AC pipes with low consequence of failure

Table 1: Consequence of Failure Rating Matrix for Roads (From Hu 2013a)

Once your inventories of high, medium, and low consequence pipes are established based on possible failure modes, each inventory of pipes is assessed differently and separately. The evaluation approach for each inventory of pipes from the WRF report is briefly presented below.

High Consequence of Failure Pipes

This is a category in which a failure of an AC main is unacceptable, and the strategy to manage this inventory of pipe is *failure prevention*. Since this is a critical pipe for a system, a utility should be willing to expend more resources on failure prevention, so some level of monitoring and testing of the pipe condition may be appropriate. The level of detail and expense can vary for the varying pipes, depending on their perceived criticality and likelihood of failure. Since these pipes are critical the utility should have a contingency plan in case a failure occurs. Appropriate utility personnel should be familiar with the plan and all necessary equipment, repair components, and supplies should be available and ready for use. Also, valves associated with this pipe should be located in the field and their functionality fully understood, and redundancy considered for any pipe in this category. Perform a hydraulic analysis to check if service can be maintained from other sources and/or other pipes in the network. If not, consideration should be given to installing another water line (ideally along a different route and from a different source), and this redundancy will decrease the criticality of the pipe (will reduce the consequences upon failure of the pipe in terms of water service outages). Also, undertake a preliminary assessment of the likelihood of failure of any critical pipes within the foreseeable future. Specifically, the preliminary risk assessment looks at factors that affect the deterioration of the pipe and factors that affect loading on the pipe. From this assessment, the expected remaining lifespan of the pipe can be estimated, given estimates of the remaining effective wall thickness of the pipe and operating pressure. Charts are provided in the *Guidance Manual* to make remaining life estimates of different diameters of AC pipe, from 100 mm (~4-inches) to 400 mm (~16-inches).

If assessment of the pipe indicates that a critical pipe also appears to have a high probability of failure, then rehabilitation and/or replacement (renewal) of that line should be considered a high priority. If renewal of the line does not appear immediately necessary, there are procedures provided in the *Guidance Manual* to estimate when the pipe should next be assessed so that breaks are prevented.

Medium Consequence of Failure Pipes

Pipes in the *medium consequence* category are similar to the *high consequence* category, except that one or two failures might be tolerated. Water mains in this category tend to be mid-size in diameter. To manage pipes in this category, the recommended steps and action plans are similar to the high consequence pipes, but generally the evaluation for redundancy can be skipped. Similarly, condition assessment/pipe sampling activities could probably be postponed until the main is considered to be at a medium risk of possible failure, or conduct detailed condition assessment/pipe sampling after the first failure.

Low Consequence of Failure Pipes

This is a category of pipe in which a certain number of failures or a certain frequency of failures is tolerable and so the strategy for these pipes should be *failure frequency management*. Water mains in this category tend to be relatively small diameter and are the often first to experience deterioration failures due to their relatively thinner wall making them more susceptible to failure. Elaborate preliminary risk assessment is not usually done for individual AC water mains in this category. Instead, the preliminary risk assessment should be done collectively for AC water mains in this category, for example on a neighborhood or cohort scale.

The most important factor in managing these pipes is to have a failure record of pipes at your utility, including the low consequence AC pipes, and assess the trends in failures over time and over different pipe cohorts. From these data the utility can establish an appropriate rate of failure associated with their pipes that is acceptable and excessive. Various statistical methods and approaches are available to manage such pipe and establish what break rate is excessive and should result in a pipe being considered for renewal. However, empirical data have indicated that one of the best predictors of a future break on a pipe is a past break on the pipe. This is understandable since the pipe is breaking in response to some type of environmental stress, and the symptom of excessive stress is a break on the pipe. While not specific to AC pipe, one utility has had great success using a criterion of 3 breaks in a 1,000 feet of pipe (Tata & Howard 2011). When a given piece of pipe reaches this threshold failure rate, it is targeted for renewal.

Due to the costs and limited benefits, condition assessment or sampling of the AC pipes in this category may not be warranted, however, opportunistic testing is often cost-effective. In addition to a pipe sample, a soil sample from the site of the break repair should be obtained for analysis.

The first step for managing water mains in this category is to ensure that data on both the inventory and the failures in the water main network are being collected and managed. Even if condition assessment work and sampling of these pipes is not routinely done, opportunistic data collection should be done when there are breaks requiring exposure of the pipe or when making new service connections on a pipe. During these events data should be noted that is available for noting.

Finally, a variety of statements have been made as to the expected “design life” of AC pipe, and some utilities have managed their AC pipe based on its “design life.” In terms of expected design life reported in the literature, the range varies from 30 years to presumably 100 years if one considers the name of the pipe in an advertisement for the pipe (“Century Asbestos-Cement Pipe”) (Keasbey & Mattison 1953). However, “design life” is strictly a guess in regards to AC pipe since no “design life” was specified in the standards, nor did one seem to have been anticipated when the standards for AC pipe were under development (AWWA 1953) nor was the term “design life” used the final standards established by AWWA. Based on typical engineering design considerations for public works, it has been taught in college-level engineering classes that most public works projects should be designed for at least a 50-year life, but this doe s not mean that what was built should be expected to fail at 50 years of age. The point being, use of age alone as a method to manage AC pipe, or any pipe for that matter, is poor engineering practice. Age is a relatively easy metric to use, but it has little relationship to the factors that actually impact pipe life.

ONGOING RESEARCH

The WRF has two ongoing research projects focused on AC pipe. The first, “Environmental Impact of Asbestos Cement Pipe Renewal Technologies,” is funded by the USEPA and is close to completion. The primary objective of this project, being conducted by John Matthews from Battelle Memorial Institute is to provide drinking water utilities with reliable performance, cost, and environmental data relating to AC pipe renewal practices. Utility practices with regards to AC pipe were examined and were found to vary state by state and from utility to utility. Most utilities preferred to abandon AC pipe in-place when possible or replace it by excavating. Although other methods of AC pipe renewal exist, such as cured-in-place pipe (CIPP), pipe reaming, and pipe bursting, utilities were hesitant to employ them based on their understanding and interpretation of the National Emission Standards for Hazardous Air Pollutants (NESHAP) and state regulations. Real-world demonstration and evaluation of two rehabilitation technologies was conducted in Florida (pipe bursting) and Nevada (CIPP).

The report will serve as a practical reference for water utilities to use when planning future rehabilitation projects on AC pipe in their distribution system. The background on Federal/State/Local regulations provides a clearer picture of what is typically allowable. In addition, the document contains detailed case studies of practical AC pipe rehabilitation projects and an idea of what types of data can be collected to ensure the projects are environmentally safe. Based upon the results of the air samples collected at each site, neither pipe bursting nor CIPP lining of AC pipe was found to have a negative impact on the surrounding air environment or the heath of the workers performing the work. Overall, the results from the soil samples collected at each site indicate only trace amounts of asbestos in the soil surrounding the pipe. With no increase in asbestos following the completion of the renewal activities (especially in the case of pipe bursting) it was determined that neither renewal method adversely impacted the soil environment. The results from the water samples collected from each site showed that the renewal technologies had no negative impact on the water quality and, in one instance, actually improved it. Therefore these technologies did not have an adverse impact on the water environment.

The second project, “Development of an Effective Management Strategy for Asbestos Cement Pipe,” is co-funded by WRF and the East Bay Municipal Utility District (EBMUD). This project is also nearing completion. This study of AC pipe performance could be particularly beneficial to the water industry. It analyzed more than 22 years of pipe break data to test various hypotheses regarding break causes. Where correlations have been found, analyses were performed to see which were causative and which appeared to simply be cross-correlations. Various statistical tests were also applied to gauge the significance of different factors. AC pipe samples were also extracted and tested using a variety of laboratory methods. Results were compared with historical pipe performance to see which variables appeared to be the better predictors of future pipe breaks. The methodology used in this study should be applicable to many drinking water system. The methodology laid out in the report could serve as a model for water utilities in investigating AC pipe breaks and managing their related AC pipe inventory.

CONCLUSIONS

Water utilities with significant inventories of AC pipe are generally concerned that this pipe is at significant risk of imminent and repeated failures within the next handful of years. There are instances where this is the case, or where the AC pipe accounts for the majority of recorded pipe breaks at a given utility. However, at other utilities AC pipe is frequently found to have significant predicted remaining life, measurable in multiple decades, in which case management of the pipe can proceed along the lines of risk considerations without unduly targeting an otherwise serviceable pipe for early replacement simply because it is an AC pipe. Tools are available, with relatively nominal investment, to assess the predicted remaining life of the pipe, and risk factors likely to contribute to earlier failure of the pipe are known and can largely be assessed. The pipe can be managed to take advantage of remaining life where that exists. Finally, while at this time the preferred method of renewing the pipe is costly removal, offsite disposal, and replacement, more cost effective methods of pipe renewal may be more routinely allowed in the future. It is hoped that the results of case studies, ongoing research, possible clarification/interpretation of NESHAP requirements specific to asbestos materials, and the interest and work of water utilities will move the industry forward with safe and effective renewal methods applicable to this pipe – when the pipe really requires renewal.

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A Comparison Study of Water Pipe Failure Prediction Models Using Weibull Distribution and Binary Logistic Regression

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Abstract

The aging water infrastructure is one of the main causes of the increasing water pipe breaks nationwide, resulting in significant social and economic costs. To sustain water infrastructure assets in good condition, it requires a proactive decision making process for capital improvement project planning, maintenance, rehabilitation and replacement. Because physical inspections involve extensive labor and financial resources, it is not always a feasible option to cover entire water pipe inspection and condition assessment. This paper presents two statistical water pipe failure prediction models developed for water system managers to assist their long-term decision making process using Weibull distribution and binary logistic regression. The models presented are developed based on water pipe failure data from approximately 100 years old cast iron pipes from a municipality in the Midwest, United States.

Keywords: Weibull distribution; Binary logistic regression; Water pipe failure; Prediction model; Cast iron.

1.0 INTRODUCTION

According to ASCE (2013), the main cause of the continuously increasing number of yearly water main breaks in the United States is the aging infrastructure; there are approximately 240,000 water breaks per year from nearly 1 million miles of water pipes. Despite the fact that water pipes have an expected design life of 50 years, the nation’s water pipes’ average age is approaching or already exceeding the initial design life. The mechanism of water pipe breaks is influenced by factors such as: operational, physical and environmental. These factors vary by local condition, construction material, quality of construction, local soil properties, operation and maintenance (O&M) practices, water quality, and flow characteristics. Thus, a single generalized water failure model, which fits nearly all systems, is not plausible.

Due to the fact of aging water pipes, there is a need to take action towards rehabilitating and replacing existing water infrastructure. Condition assessment is used to make decisions about whether or how to renew a specific pipe. Although, the traditional physical inspection of pipe segments is an effective condition assessment method, it requires extensive resources to be carried out.

Statistical modeling is an alternative and more economically feasible method that involves water pipe failure data; and that can help water utility managers in planning (budgeting) rather than making a specific engineering decision on a segment of sewer system rehab and replacement. Many statistical models have been developed that rely on available data such as pipe, trench, and historical water break data to predict the break rate of water mains in a network (Nishiyama & Filion, 2013). Due to the limited data availability, reliability, and collection methods, it becomes challenging to solely rely on statistical prediction model in lieu of physical inspection.

The main purpose of this study is to present a comparison study of two statistical failure prediction models to estimate remaining life and predict future failure pattern of cast iron pipes. Two models using Weibull distribution and binary logistic regression were developed based on water pipe break data obtained from a large size municipality (approximately 250,000 residents) in the Midwest. The results of the two modeling techniques show similar failure pattern from the analyzed pipe segments. The findings of this study may assist long-term decision making process for capital improvement project planning and operation and maintenance process for water utility managers.

2.0 REVIEW OF RELEVANT LITERATURE

Various studies have developed statistical models in an attempt to predict water pipe failure. Statistical break models are developed based on water break data and predict future pipe breaks by using several variables available from the data and developing predictive equations (Kleiner & Rajani, 2001). The most often used predictive models are regression type models, time linear models, time exponential models, proportional hazards models, generalized linear models, and data driven models (Yamijala, 2007).

The first statistical model developed in this study assumes that the time between installation and the first break is described by a Weibull distribution. A Weibull distribution is described by both a hazard and a survival function. Cox (1972, as cited in da Costa Martins, 2011) proposed the proportional hazards model which is able to identify factors that affect pipe failure by using the partial likelihood function (Cox, 1972; da Costa Martins, 2011). Andreou (1986, as cited in Rostum, 2000) uses Cox's proportional hazards model to predict the failure probability of individual pipes in two large water utility networks in the United States. The proportional hazards model describes the break rate as a function of time (Andreou, 1986, as cited in Rostum, 2000). A model that follows two probability distributions was proposed by Mailhot, Pelletier, Noel and Villeneuve (2000). The model is based on a two-parameter Weibull distribution for the time elapsed between the installation of the pipe and the first break, and a one-parameter exponential distribution for the time elapsed between subsequent breaks. The parameters of the model were obtained by maximization of a log-likelihood function; after the parameters were obtained, the number of pipe breaks was estimated. To link multiple explanatory variables (such as pipe diameter, length, number of breaks, etc.) to the time elapsed between breaks within a period of observation, LeGat and Eisenbeis (2000) proposed the Weibull proportional hazard model (LeGat & Eisenbeis, 2000; Alvisi & Franchini, 2010). The model combines the proportional hazard model with the Weibull power law, and these two can be considered analogous (LeGat & Eisenbeis, 2000; da Costa Martins, 2011). LeGat (1998, as cited in Rostum, 2000) also studied the expected number of pipe breaks in an irrigation system in France, where the Weibull proportional hazard model was used and the expected number of pipe breaks was predicted.

The second statistical water pipe break prediction model developed in this study uses binary logistic regression. Regression models have been used extensively to predict the probability of future breaks and estimating the number of subsequent breaks in water networks. These predictive models are developed based on simple and multiple linear, nonlinear; logistic, Bayesian, and other type of regression analysis.

One of the earliest models was developed by Shamir and Howard (1979), who determined the optimal time for pipe replacement by assuming that future break patterns will be similar to the existing break history. The model, based on an exponential regression equation, predicts the number of pipe breaks per year per 1000 ft.; however, it has several limitations because the only predictive factor was pipe age. Other factors like pipe diameter, soil conditions, material of pipe, and length can have serious effect on pipe's structural integrity and should be included in the model.

Another statistical break model, based on Shamir and Howard's approach, was developed by Kleiner and Rajani (2002), who found a relation between multiple variables and pipe break rates. They analyzed how time-dependent factors such as cumulative length of replaced mains, rainfall deficit, freezing index and cumulative length of cathodic protection of cast iron and ductile iron pipes influenced the break rate of those pipes. In addition, Clark, Stafford and Goodrich (1982) developed a regression model to predict the number of subsequent breaks after the first break occurs in a water distribution network. They developed a second regression equation that predicted the time until the first break occurs on pipe segments that have no break history. The results showed that if covariates such as residential and industrial development were added to the equation, the obtained models yielded poor results, with low R^2 values (Clark, Stafford & Goodrich, 1982; Rostum, 2000).

A multiple regression model was proposed by Wang, Zayed and Moselhi (2009), who predicted the annual break rates of a large Canadian municipality by developing five multiple regression models. The models used data on the pipe's material, diameter, length, installation year and depth of burial. However, the model cannot predict when the next failure is going to occur for a specific pipe.

Binary logistic regression was implemented by Koo and Ariaratnam (2006) for assessing the extent of deterioration of a sewer system. The findings can be used in a practical manner to improve decision-making process and resource allocation for capital improvement projects.

3.0 WATER PIPE DATA DESCRIPTION

Water pipe data was obtained from a city founded over 200 years ago and incorporated as a city over 170 years ago. The age of the oldest water pipe is not clearly known, but the city has many miles of water pipe over 100 years (see Figure 1). The total length of the water network installed between 1900 and 2013 is 1,164 miles and serves a population of approximately 250,000. The water network service area covers 122 square miles in mostly urban area. The predominant pipe material in the early stages of installation was cast iron, followed by ductile iron and PVC. Figure 1 represents pipe installation miles and materials per decade.

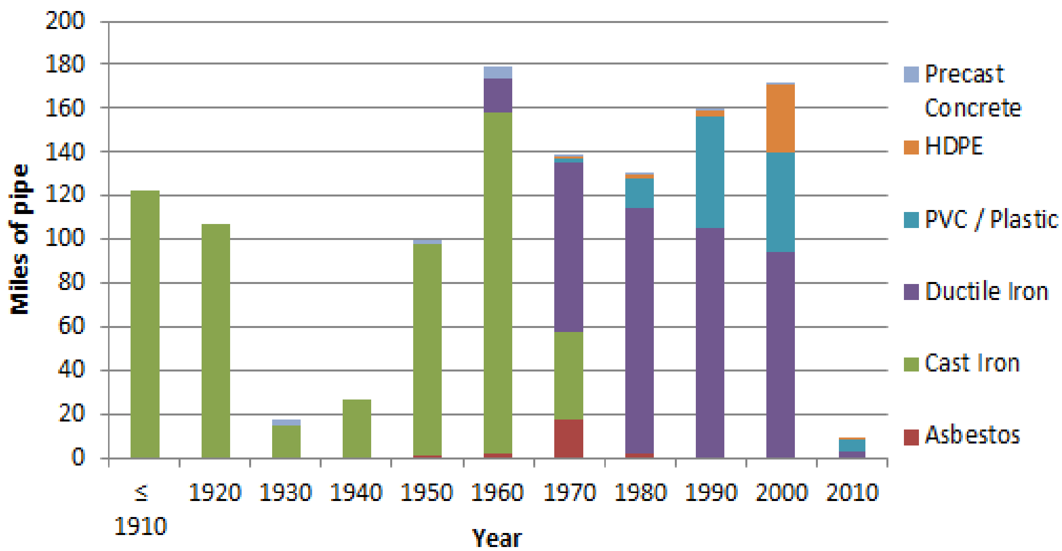

Figure 1. Miles of pipe installed by material (data from the 2014 Asset Management Report of the municipality)

As seen in Figure 1, the majority of the water system is composed of cast iron pipes with a total installed length of 565 miles, followed by ductile iron pipes with a length of 409 miles. HDPE, asbestos cement, precast concrete, PVC and steel pipes together have a total length of approximately 75 miles. Similarly, the majority of water main failures occurred in cast iron pipes, followed by ductile iron pipes. Water pipe failure data is available starting from 1974 to 2013. Accordingly, a total of 3,106 failures occurred in this period, from which approximately 2,617 (85 %) in cast iron pipes and 397 (12%) in ductile iron pipes.

Moreover, the data analysis of pipe diameters showed that approximately 95% of the total water network is comprised of pipes with the diameters of 6”, 8”, 12”, and 16” as shown in Figure 2.

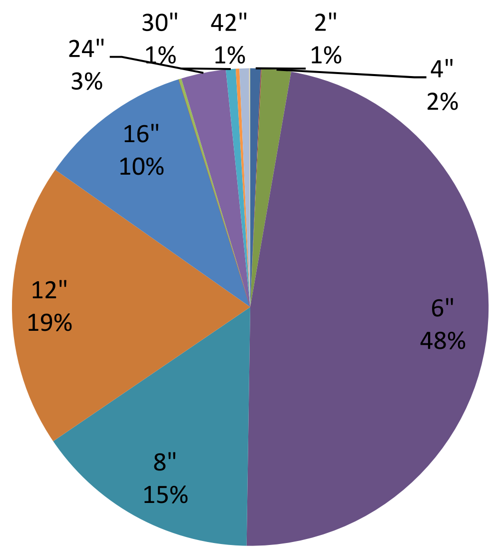

Figure 2. Water network analysis by pipe diameter (data from the 2014 Asset Management Report of the municipality)

4.0 WEIBULL DISTRIBUTION MODEL

The Weibull distribution model predicts statistical failure probability based on existing pipe failure data. The model uses Weibull distribution to plot graphical solution demonstrating a statistical trend of failure in the future. For this study, pipe failure is defined when the first break occurs, regardless of subsequent breaks thereafter on the pipe segment. Every pipe segment is treated as a single data point that has a unique identification number, length, and diameter. Local physical, operational, and environmental condition data such as depth of cover, corrosiveness of backfilled soil, groundwater, water quality, and construction quality were not captured in the data. The data includes installation year, break year, diameter, segment length, and types of material.

The Weibull distribution model for this study selects installation year to calculate pipe age, and failure year to calculate time interval between installation and the first event of break (time to failure). For this paper, the selected data for developing the predictive model includes only cast iron pipes installed between 1900 and 1910, with the diameters of 6", 8" and 12". The model uses actual failure data, which is only available from 1974 to 2013, for the cast iron pipes installed between 1900 and 1910. The model predicts failure probability during a period of 87 years of service, from 2013 to 2100. In developing the predictive model, only failure data from 1974 to 2007 was included. During this period, a total of 205 pipe failures were recorded, representing 74% of the total pipe failures recorded for cast iron pipe segments of 6", 8", and 12" installed between 1900 and 1910. Model validation was achieved by comparing actual failure data from 2008 to 2013 with the predicted failure rates for the same period (see Table 3).

A two-parameter Weibull distribution was used to develop the predictive model using equation (1):

$$f\,(x, \lambda, k) = \frac{k}{\lambda}\left(\frac{x}{\lambda}\right)^{k-1} e^{-\left(\frac{x}{\lambda}\right)^{k}} \quad (1)$$

Where:

$\frac{k}{\lambda}\left(\frac{x}{\lambda}\right)^{k-1}$ = Hazard function or failure rate of the Weibull distribution
$e^{-\left(\frac{x}{\lambda}\right)^{k}}$ = Survival function of the Weibull distribution
k = Shape parameter of the Weibull distribution
λ = Scale parameter of the Weibull distribution
x =Time to failure

The two parameters of the Weibull distribution, k and λ, were obtained by using the rank regression method (Nelson 2005). The shape parameter, k, is an indicator of whether the failure rate is increasing (k>1), decreasing (k<1), or constant (k=1). The slope parameter, λ, is a measure of the scale, or spread, in the distribution of data.

The lack of recorded failure data prior to 1974 has the potential to alter the obtained parameters of the Weibull distribution, which can result in a skewed graphical representation of the failure probability prediction of the cast iron pipes selected for this study. For pipe segments installed in 1900, the earliest recorded failure was in 1974. The authors assumed that the first time span to reach failure of cast iron pipe segments installed between 1900 and 1910 is 74 years. This assumption may potentially overestimate the probability of failure in Figure 3.

The results show an increasing tendency of the failure rate. The Weibull analysis parameter estimates for cast iron pipes installed between 1900 and 1910 were determined to be as follow: k= 4.96 and λ=142.35. Figure 3 presents the Weibull plot for cast iron pipe failure model.

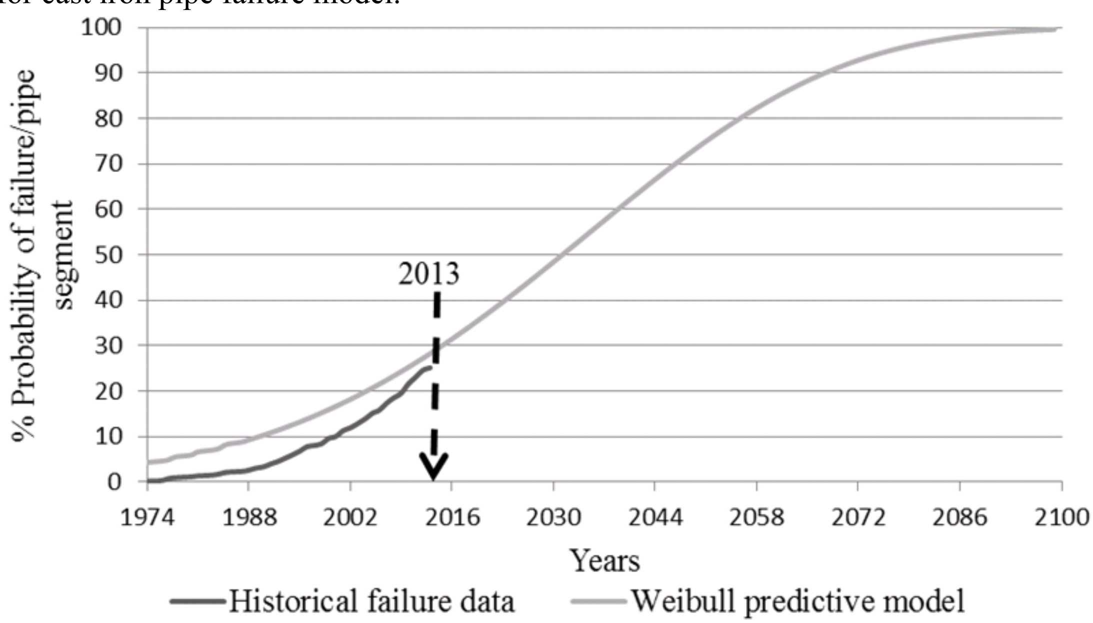

Figure 3. 6”, 8” and 12” cast iron pipe (installed between 1900 and 1910) failure model

5.0 BINARY LOGISTIC REGRESSION MODEL

The second predictive model uses the binary logistic regression method. The model examines how various independent parameters influence a dependent dichotomous variable, estimating the probability of an event's occurrence (Anderson, 1982). Binary logistic regression can be thought of as a method similar to multiple linear regressions, but the binary logistic regression can treat categorical parameters as dependent variables. In this case, the dependent variable is a dichotomous parameter, with the categories failure or no failure. By developing a model using binary logistic regression, the probability of a pipe segment to fail at any given time can be predicted. The selected data for developing the predictive model includes cast iron pipes installed between 1900 and 1910, with the diameters of 6", 8" and 12". Two independent variables were considered in the development of the predictive model: pipe diameter and pipe age. For developing the predictive model, table 1 presents the categorization of the data based on the pipe's diameter. The dependent variable is a binary variable coded either 0 (no failure) or 1 (failure).

Table 1. Data categorization for developing the predictive failure model

Independent Variable	Category 1	Category 2	Category 3
Pipe diameter	6"	8"	12"

A total of 1,106 pipe segments were available for the data analysis, from which 278 pipe segments had recorded failure during 1974 and 2013. For this model too, only failure data from 1974 to 2007 was considered (approximately 74% of the total pipe failures recorded). The binary logistic regression model validation was obtained by comparing the actual failure data recorded between 2008 and 2013 with predicted failure probability rates in Table 3.

SPSS™ was used as a statistical analysis tool. To link the dependent variable to the set of explanatory variables, the logistic transformation is used. According to Dayton (1992), the logistic coefficients can be estimated by using the maximum likelihood principle, as opposed to the least squared principle usually used in linear regression. Equation (2) can be generated using the constant and coefficients from the multiple logistic regression model result. The equation is presented below with f(x) representing the log-odds of a pipe segment to fail:

$$f(x) = \log_e(P) = \log\left[\frac{P}{1-P}\right] = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \beta_3 x_4 + \cdots + \beta_n x_n \quad (2)$$

Where:

$\frac{P}{1-P}$ = odds of a pipe to break

$(x_1, x_2, \ldots, x_n)$ = independent variables

β_0 = constant

$\beta_1, \beta_{2,\ldots,}\ \beta_n$ = variable coefficients

Accordingly, by applying equation (3), the probability of a pipe to fail can be obtained:

$$\text{P r}(\text{Y}=1|\text{x}) = \text{P}(\text{x}) = \frac{e^{f_x}}{1+e^{f_x}} = \frac{1}{1+e^{-f_x}} \quad (3)$$

Where:

Pr = the probability of event Y (pipe break)
x=time to failure

Independent variables (pipe age and diameter) can be tested by their significance in the model. Two statistical tests, the Wald test and the log likelihood tests, are commonly used. The Wald test is to compare the significance of one independent variable to another, and is the square of the "z" test of each logistic coefficient. The Diameter parameter showed the lowest value of Wald test result and the statistical significance, p, showed a higher value than 0.05. As a result, the diameter parameter is not statistically significant for the model; therefore, there is no significant difference between the failure rates of pipe segments with the diameters of 6", 8" and 12". On the contrary, age is a statistically significant parameter in the model. In addition, the log-likelihood test was used to test the statistical significance of the independent variables by using both the Chi-square distribution and corresponding p value. The log likelihood test results are presented in Table 2.

Table 2. Log likelihood test of statistical significance of independent variables

Independent Variables in the Nested Model	Chi-square	D. F.	Sig.	Critical Value (95 %)	p value for critical level	Results
Age	10.35	1	0.001	3.841	0.05	Age is a significant variable
Age, Diameter	128.752	1	0.375	3.841	0.05	Diameter is not a significant variable

After obtaining the values of the binary coefficients, equation (2) was used to obtain the log-odd ratios (log (p/(1-p)) used to plot probability of failure in Figure 4. Despite low statistical significance of the diameter variable in the test, the model includes it because the significance varies depending on the data used in the model. Probability of failure can be obtained as follows:

$$f(x) = \log_e(P) = \log\left[\frac{P}{1-P}\right] = 15.25 - 0.131 * \text{Age} - 0.011 * \text{Diameter}$$

Equation (2) was used to calculate the probabilities of failure at given time to failure periods. Figure 4 represents the predicted probability of failure at a given time for cast iron pipe segments installed between 1900 and 1910 with the diameters of 6", 8" and 12", modelled with the binary logistic regression method.

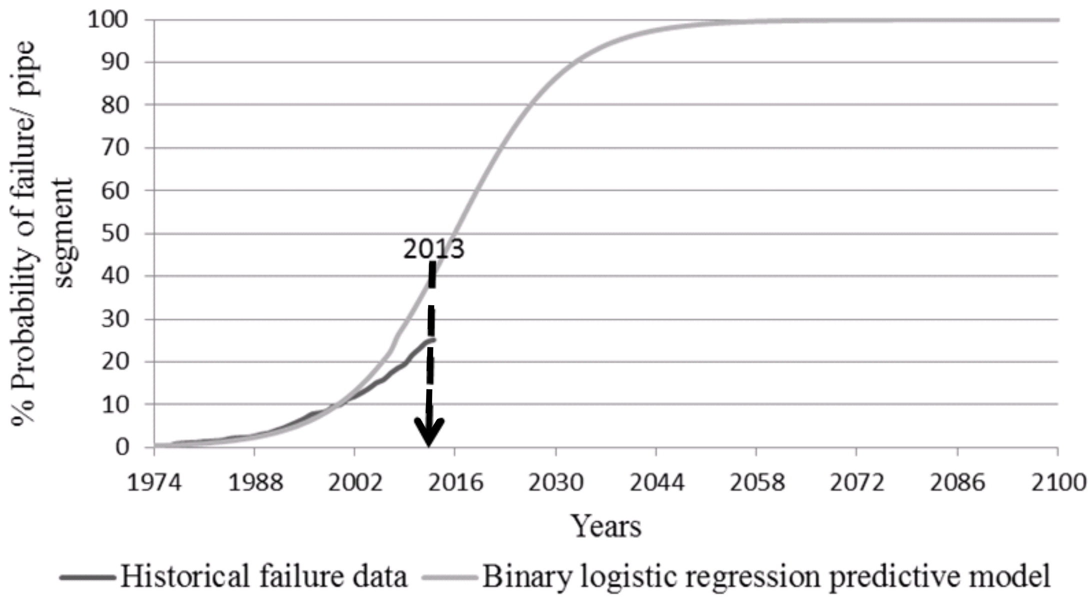

Figure 4. 6", 8" and 12" cast iron pipe (installed between 1900 and 1910) failure model using Binary Logistic Regression

6.0 DISCUSSION

This section describes the comparison between the two models; 1 % of probability of failure means that there is a chance of 1 % that pipe segments can be broken out of the entire water pipe network. Figures 3 and 4 show the cumulative probability of failure from 1974 to 2100 for cast iron pipes installed between 1900 and 1910 (diameters 6", 8" and 12"). Based on the modeling assumptions, both graphs reached 100 % probability of failure of a pipe segment (assumed as 200 years of maximum life cycle from 1900 for both statistical models).

The actual water pipe segment failure data reveals that approximately 25.13% of the cast iron pipes with the diameters of 6", 8" and 12" installed between 1900 and 1910 had failed between 1974 and 2013. The models were developed using cast iron pipe segment failure data from 1974 to 2007. The validation of the models was made by comparing the actual failures for the remaining 6 years of actual data, from 2008 to 2013, to the results of the two statistical models' predicted probability of failure. The models show a tendency of overestimating the actual failure rates. The overall average difference between the actual failure data and the Weibull prediction model is -2.71% and it ranges between -3.89% and -1.63%. For the binary logistic regression model, the overall average difference is -11.08% and it ranges between –15.43% and -7.64%. Table 3 summarizes the comparison between the actual failure and predicted probability of failure of both models.

Table 3. Actual failure data vs. predicted probability of failure from Weibull and Binary Logistic Regression models

Year	Actual Failure Data [% Failure]	Weibull Model Predicted Probability [%Failure]	Actual-Weibull Prediction [%]	Binary Logistic Regression Model Predicted Probability [%Failure]	Actual-Binary Logistic Regression Prediction [%]
2008	18.55	22.44	-3.89	26.19	-7.64
2009	19.55	23.36	-3.81	28.80	-9.25
2010	21.63	24.30	-2.67	31.56	-9.93
2011	23.08	25.26	-2.18	34.46	-11.38
2012	24.62	26.24	-1.63	37.47	-12.86
2013	25.16	27.25	-2.09	40.59	-15.43
Average			-2.71		-11.08
Max. Difference			-1.63		-7.64
Min. Difference			-3.89		-15.43

Benefits of the binary logistic regression model include allowing the introduction of multiple explanatory variables to improve the model's reliability and predictive capability. However, the validation results show that the Weibull model's failure probability prediction is more accurate than the Binary Logistic Regression model. A major reason for these results can be caused by the small sample size of pipes that failed in the population (278 pipe failures out of a total population 1,105 pipe segments). Bergtold, Yeager and Featherstone (2011) showed that a small sample size can bias the parameter estimates of the model. Nemes et al (2009) also demonstrated that logistic regression overestimates the odds ratios in studies with small to moderate samples size, and the fit is better for continuous variables than for discrete ones. Table 4 present a summary of the predicted probability of failure from 2015 until 2100.

Table 4. Predicted probability of failure from Weibull and Binary Logistic Regression models in 200 years life cycle

Year	Weibull Model Predicted Probability [%Failure]	Binary Logistic Regression Model Predicted Probability [%Failure]
2015	29.32	47.03
2020	34.86	63.09
2025	40.83	76.69
2030	47.14	86.37
2035	53.64	92.42
2040	60.18	95.91
2045	66.57	97.84
2050	72.65	98.86
2055	78.25	99.41
2060	83.23	99.69
2065	87.51	99.84
2070	91.04	99.92
2075	93.83	99.96
2080	95.93	99.98
2085	97.45	99.99
2090	98.48	99.99

2095	99.15	100.00
2100	100.00	100.00

Based on the analysis of the actual failure data and predicted probability of failure, the two statistical models are valid tools for decision makers and water pipe managers, who may utilize the prediction models as a long-term capital improvement project planning tool.

7.0 CONCLUSIONS

Statistical modeling has been used extensively as an attempt to model the failure likelihood of water infrastructure. Most often the number of water main breaks is considered as an indicator of the actual physical deterioration status of a water network. However, statistical models offer only a theoretical prediction of the actual condition of the water system, and are developed solely with the available water break data which is often insufficient and incomplete.

This study developed two statistical water pipe failure prediction models to predict the probability of failure based on the actual failure pattern of water pipes in a municipality in the Midwest, the United States. Cast iron pipe segments installed between 1900 and 1910, with the diameters of 6", 8", and 12" were selected for the model development. Weibull analysis was used to model the probability of failure by estimating two calibration parameters of a statistical water pipe failure prediction model. Binary logistic regression was the second statistical modeling methodology predicting probability of segment failure of cast iron pipe. Pipe segment failure data, age, and diameter were used to determine the logistic coefficients.

The comparison of the two models indicates that the two statistical pipe failure prediction models track closely the patterns of the actual failure data, and show asymptotically increasing probability of failure. The model and comparison study results provide a holistic view of water pipe failure pattern to water utility owners to assist the long-term capital improvement planning process and asset management.

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Where are the Hot Zones: Prioritization with Historical Pipe Break

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Abstract

This paper introduces the use of cluster analysis in pipeline asset management. It utilizes the pipeline geographic information and the historical pipeline break data to test if the pipe breaks are randomly distributed in space and time, and whether any hot zones can be detected. The results contribute to risk analysis and decision-making under a pipeline asset management program. When additional pipe information is available, the cluster analysis can eliminate the impact of different pipe attributes to break rates and reveal the existence of the hot zones due to the complex environmental and hydraulic conditions. The application is demonstrated on a medium-size water distribution network in North America.

INTRODUCTION

Many water utilities are challenged with aging infrastructure, growing population and limited funds. In the United States, the drinking water infrastructure has more than one million miles of pipes approaching the end of their useful life. Between 2011 and 2035, the total cost of replacing pipes at the end of their useful lives is estimated to be more than $1 trillion dollars (AWWA, 2012). To put the number in perspective, this equates to over $8,000 dollars per household based on 2014 census results. The combination of the great need in water infrastructure management and the limited resources has driven the development of pipeline asset management.

A successful asset management will not only improve the decision making process throughout the life cycle of a pipeline, but also extend the pipeline life. Risk-based prioritization is a vital part of asset management for increasing the efficiency of decision making and maximizing the return of the investment. Risk is commonly calculated as multiplication of the likelihood of pipe failure and the consequence of failure. The higher the risk score is, the higher priority an asset should be given. The consequence of failure can be assessed on environmental, economic and safety standpoints, while the likelihood of failure depends on the pipe's structural deterioration and remaining useful life.

Many factors affect the deterioration of water and wastewater pipelines. There are four major categories of factors: 1) structural or physical variables; 2) external or environmental variables; 3) internal or hydraulic variables; and 4) maintenance variable (Rostum, 1997). A pipe failure can be the result of one factor alone or a combination of factors. Repair or replacement of a section of failed pipe may reset the structural condition, but not necessarily resolve the underlying unfavorable environmental or maintenance issues. For example, a highly corrosive environment has caused multiple leaks on the steel pipes in the vicinity. Replacing the corroded pipes with new steel pipes will not change the soil pH value or alter the resistivity. Future failures could occur again to the new pipes under the same environmental effect.

The pipe failure history is a significant aspect for the prediction of future failure trend and for determining the underlying causation of pipe failure. Goulter and Kazemi (1988) observed the temporal and spatial clustering of water-main breaks and suggested that some pipe breaks can be accounted for by the repair work on previous breaks. Pipes in the vicinity of a pipe break often have the same age and same material. The likelihood of their failure might be higher as they are typically exposed to the same external environment and internal hydraulic operation.

This paper focuses on a risk based pipeline prioritization featuring a cluster analysis model which utilizes the pipelines geographic information and the pipe break data to detect pipe break hot zones over space and time to determine if any hot zones exist. Whether a pipeline belongs to a hot zone becomes a data point in risk analysis and provides valuable information for the planning of maintenance and condition assessment activities. A case study has been conducted for a medium-size water distribution network in North America.

METHODOLOGY

Cluster analysis is a data reduction tool that creates subgroups that are more manageable than individual element (Burns, Richard 2009). The water pipeline network can be organized into different hot zones by analyzing the recorded water main break data. In this paper, a hot zone and significant clusters are used interchangeably. A hot zone is defined as a cluster inside which the observed pipe break rates are significantly higher than the expected pipe break rates.

The level of the sophistication of the cluster analysis model depends on the availability of the information.

Level 1: Pipelines geographic information and historical pipe break record (location only)

At this level, a two-dimensional spatial scan statistic is performed to identify clusters. Developed by Martin Kulldorff, the scan statistic is widely used in epidemiology for geographical disease cluster detection (Kulldorff, 1997, 2011). A circular or elliptic window scans the map for clusters. The size of the scan window varies continuously, noting the number of observed and expected observations inside the window at each location. Each scan window is a possible candidate for cluster. The number of pipe breaks in a geographical location is assumed to follow a discrete Poisson distribution. Under the null hypothesis, the expected number of pipe breaks inside the scanning window is proportional to the total pipe length.

The pipeline spatial data, in the form of geographic information system (GIS), served as the backbone of the cluster analysis. The extent of the water pipeline network was divided into grids. Each grid element is a cluster unit in space. In general, as the size of the grids decrease, the results can provide higher precision but the analysis demands more computing power. Each grid carries two attributes: the total length of pipeline within the grid and the associated historical break information.

When the pipe break location is recorded as the address of the nearest property or street intersection, geocoding is required to derive the geographic coordinates based on street name, street number and zip code before a scan analysis can be performed.

Level 2: Pipelines geographic information and historical pipe break record (location and time)

At this level, a three-dimensional space-time scan statistic is performed to identify clusters. A cylindrical window with circular or elliptic base, which gradually varies in size and shape, scans the map for clusters. The base of the scan window reflects the possible geographic area, while the height of the cylinder reflects the possible time period.

Similar to the grid being the unit for cluster in space, a time step, or time precision, will be chosen as the unit of cluster in time, such as year or month. By refining the time precision in the cluster analysis, it enables the capture of seasonal patterns of pipe break activities. Temperature changes are known to effect water pipe breaks. This can be valuable information in planning the maintenance activity, for example, where and when to prepare crews for high break response or postponing planned work to facilitate emergency work.

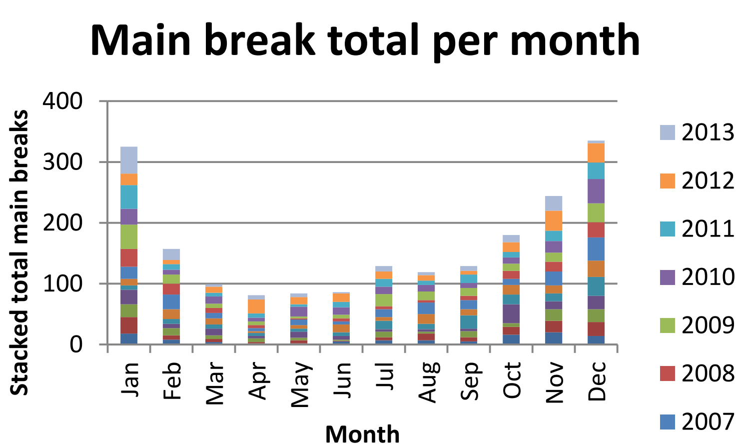

Figure 1: Utility X historical pipe break by month

Level 3: Pipelines geographic information, historical pipe break record and pipe attributes information such as material and age

At this level, the contribution of different covariates, such as pipe age and material, to the break rate may be eliminated. The past pipe break pattern can potentially reveal the existence of hot zones due to complex environmental and hydraulic conditions.

At first, the pipeline system is divided into different groups according to the attributes. Each grid, as mentioned earlier, will store information of the pipe length and historical break record for all groups separately within the grid boundaries. Then, a space-time scan statistic is performed to identify clusters.

CASE STUDY

The proposed scan analysis is demonstrated on a medium-size water utility's entire water distribution system in the United States. This utility is referred to as utility X in this paper. Until approval is received, the name remains anonymous.

A level 3 cluster analysis was selected given the following available data:

1) Pipeline geographic information system. The system was divided into 750ft by 750ft grids. This grid size allows for desired precision within reasonable computing timing.

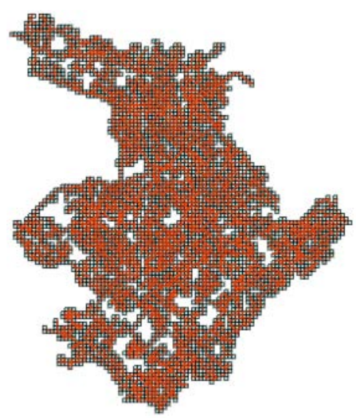

Figure 2: Water Pipeline Network Grid Map

2) Historical break records. Breaks were collected and aggregated by year from 1998 to 2012. Each break was associated with a pipe ID through spatial correlation with the aid of GIS tools. The model time step was set to be annual as the breaks are recorded.

3) Pipeline installation year and pipe material. These two pipe attributes serve as the covariates of the model.

Result

A total of 21 clusters were identified, out of which 13 clusters are considered hot zones and the remaining 8 clusters were not significant based on the computed P-value.

Table 1: Scan Analysis Partial Results

Cluster ID	P_Value	Observed number of pipe breaks	Expected number of pipe breaks	Hot zone (Significant Cluster)
1	<0.01	16	<1	Yes
2	<0.01	52	12	Yes
…	…	…	…	…
13	0.02	21	5	Yes
14	0.18	16	4	No
…	…	…	…	…
20	0.98	25	10	No
21	0.99	4	<1	No

P-value is a function of the observed sample results which is used for testing a statistical hypothesis. Any cluster with P-value less than 0.05 was considered significant, thus a hot zone. The results are shown in Table 1 above and are plotted spatially in Figure 3 below. Red shows significant clusters/ hot zones while blue identifies non-significant clusters.

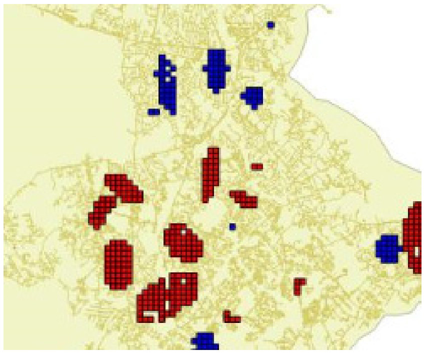

Figure 3: Result Map for Cluster Analysis

The cluster analysis is one of the factors of the risk analysis, as shown in Table 2.

Table 2: Likelihood of Failure Table

Factor ID	Name	Description	Likelihood of Failure
1	Cluster Analysis	Significant cluster	High
1	Cluster Analysis	Non-significant cluster	Medium
1	Cluster Analysis	Not a cluster	Low
…	Other Factors	…	…

CONCLUSION

Proper understanding of water and wastewater pipeline system deterioration is crucial in developing cost-effective, efficient, and successful pipeline management strategies. The cluster analysis can be tailored according to the availability of the information and provides valuable information for the planning of maintenance activities.

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Minimizing the Risk of Catastrophic Failure of PCCP in the City of Baltimore

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Abstract

With at least 28 existing prestressed concrete cylinder pipe (PCCP) failures in the water system and little history of preventive maintenance, the City of Baltimore (the City) must take significant steps to manage the PCCP in its system. This year, the City is implementing a strategic and comprehensive assessment, monitoring, and repair program for its inventory of PCCP. In this initial ten-year program, the City is strategically prioritizing pipes for inspection through a detailed risk assessment model and maximizing the inspection of high-risk pipes in the program's early stages. Condition assessment is performed for each inspection zone using a combination of acoustic, visual, and electromagnetic technologies. Acoustic fiber optic systems will also be installed to effectively monitor pipe segments for wire breaks in high-risk pipes, allowing the City to respond to damaged pipe before a catastrophic failure occurs. In parallel, the City is carrying out an outreach plan to proactively share the approach with stakeholders.

PURPOSE AND DRIVERS

The purpose of the PCCP risk management program is to implement a comprehensive inspection, monitoring, repair, and replacement program for PCCP in the Baltimore Metropolitan Water System.

The program is driven primarily by the desire to minimize catastrophic failures of PCCP – there have been at least 28 recorded PCCP failures in the water system from 1977-2010, including a heavily publicized failure in Dundalk in 2009 – as well as the need to develop a comprehensive knowledge of the inventory and condition of PCCP in the system methodically and strategically. Historically, the City has relied on reactive maintenance alone to address pipes that have failed or approached the end of their useful lives.

Implementation of the PCCP program will allow the City to effectively and knowledgeably manage the risk of PCCP failures, predict pipeline damage and failures, and take appropriate actions to repair or renew PCCP in a cost-effective manner. While eliminating all risk is cost prohibitive, the City's risk management strategy can significantly reduce the chances of a major PCCP failure by implementing corrective actions before life and property are jeopardized.

This paper will discuss the City's PCCP risk management program, the role of the program within the City's overall asset management approach, prioritization strategy, inspection and repair techniques, and proactive outreach plan to communicate the program to internal and external stakeholders.

ROLE OF PCCP IN OVERALL ASSET MANAGEMENT APPROACH

The PCCP program is part of a larger effort to fast-track strategic asset management programs for the City's aging linear infrastructure and is a significant part of the City's holistic asset management strategy.

In 2013, the City Department of Public Works (DPW) formed the Utility Asset Management Division, now called the Office of Asset Management, charged with developing and implementing preventative maintenance and asset management programs across the City's water and wastewater linear assets. Figure 1 displays the City's overall approach to reliability and risk for its water transmission and distribution system, which provides water to 1.8 million people through 4,000 miles of pipe. Though the system is in both the City and in Baltimore County, the City is responsible for maintenance of all pipes.

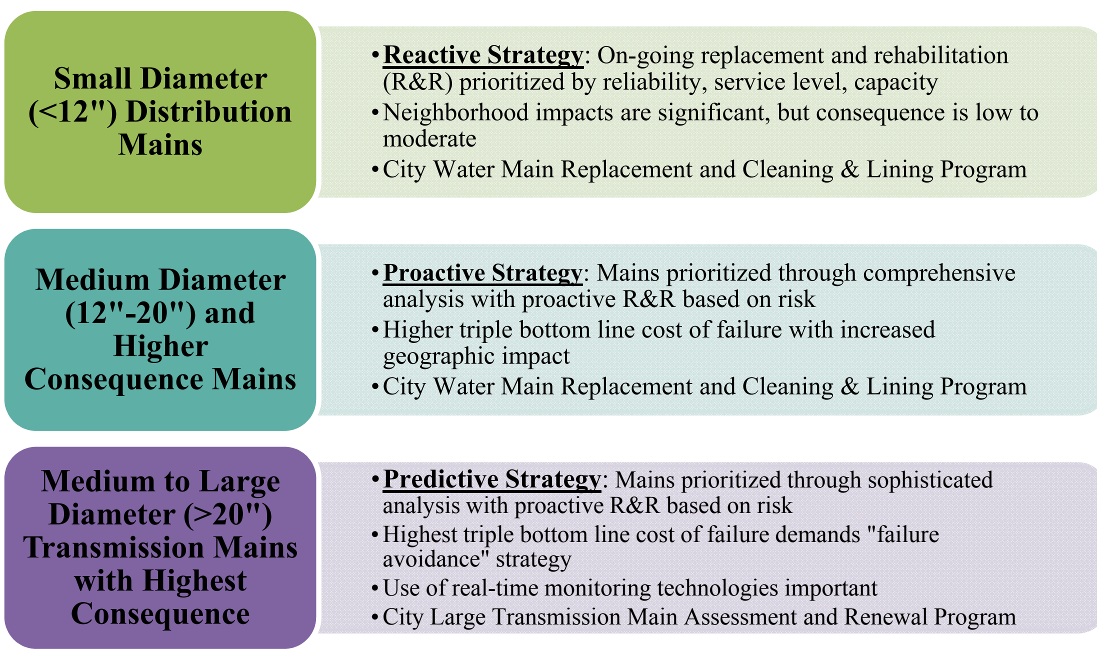

Figure 1. Overall reliability and risk matrix for Baltimore water distribution and transmission system.

Nearly all the 110 miles of PCCP in the City's inventory falls within the third category denoted in Figure 1. Reducing risk of failure of PCCP in the system requires a *predictive* approach to inspect and monitor the condition of these assets and repair or replace segments before they fail. This proactive, predictive approach is important to avoid the significant triple bottom line impacts of a single PCCP failure. These negative impacts include extended residential, commercial, and industrial customer interruptions, extensive pipe repair costs and complexity, potential for mild to moderate collateral damage to property, and risk of injury or physical harm from a catastrophic failure.

PCCP INVENTORY

The Baltimore Metropolitan Water System has approximately 110 miles of PCCP in total, with about 87 miles of PCCP ranging in size from 36-inch to 120-inch in diameter. The inventory of mains 36-inch in diameter or greater is well-known due to a 2002 City project to build a master inventory of PCCP and RCCP using as-built drawings, pipe lay schedules, and other available record documents.

According to the inventory, the predominant type of PCCP used in Baltimore is embedded cylinder pipe, comprising 61 percent of the inventory. Approximately 22 percent, or 20 miles, is constructed with Class IV wire. High strength Class IV prestressed wire, manufactured between about 1972 and 1980, has been shown to have compromised overall quality and in-service performance due to hydrogen embrittlement, and has a higher likelihood of failure than other wire classes.

The inventory of PCCP between 20-inch and 36-inch is less clear. The City has records that pipes are made of "concrete" through the GIS database, but knowledge of the type of concrete – PCCP, RCCP, *etc.* – is not well documented in the majority of cases. A detailed inventory of the 20-inch to 36-inch PCCP, similar to the work completed for the larger diameter pipes, will need to be conducted simultaneously with the long-term pipeline assessment. It is estimated that about 20 miles of PCCP between 20-inch and 36-inch exist in the water system.

The City has knowledge of many of the small segments of PCCP – short lengths of PCCP nestled in between longer segments of metallic pipe – in the water system, but this represents an incomplete inventory as well. Other small segments of PCCP may exist in the system that are not captured in existing records. The current City strategy is to replace these small segments with metallic pipe, likely ductile iron, as the opportunity presents itself when repair activities are performed nearby.

Figure 2 below presents recorded information on the number of failures of PCCP in the Baltimore Metropolitan Water System between 1977 and 2010.

About 72% of the failures recorded above occurred on Class IV PCCP. Of note is the recent transmission main failure in September 2009 in Dundalk, in the southeastern side of Baltimore, which occurred on a 72-inch Class IV main installed in 1974. The pipe failure was attributed to external heavy loading and caused flooding of more than 100 homes and commercial properties and several dozen cars in the region. The incident received significant media attention and led to City investment in developing a comprehensive PCCP inspection program.

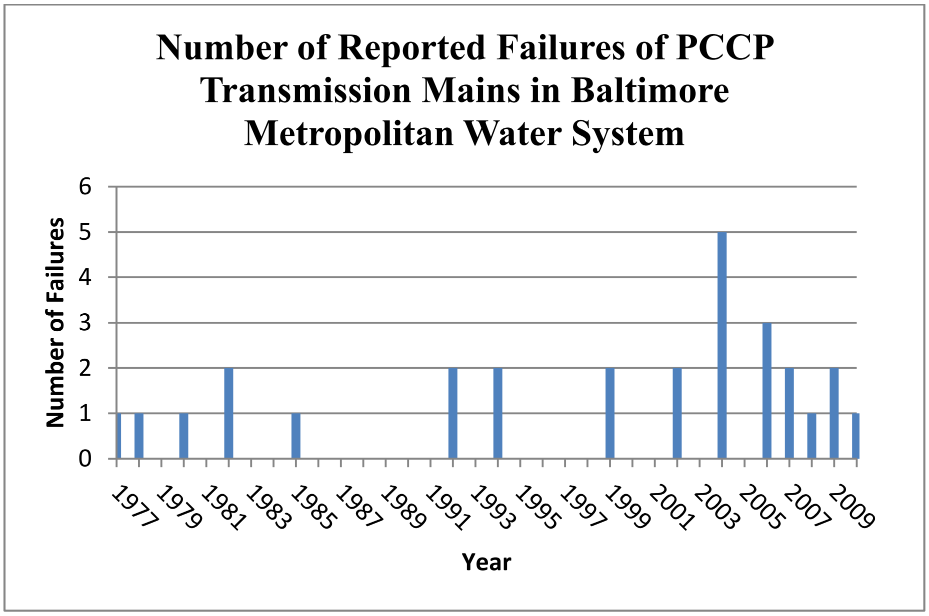

Figure 2. Recorded PCCP failures in Baltimore Metropolitan Water System.

SUMMARY OF PROGRAM STRATEGY

Figure 3 provides an overview of the strategy to assess, rehabilitate or replace, and monitor PCCP in the system.

The City prioritized PCCP for inspection through an established desktop risk assessment model. The model calculates risk scores for pipe segments based on various factors of condition and criticality and will allow the City to dedicate limited resources to inspect and manage PCCP effectively. The risk assessment currently contains a complete inventory of the City's transmission mains greater than or equal to 36-inch diameter only; the inventory of transmission mains smaller than 36-inch and of small segments of PCCP (short segments of concrete in between longer segments of metallic pipe) scattered throughout the water system is less complete.

In this risk-based approach, the highest risk segments of PCCP in the system will be inspected first. The City will conduct detailed non-destructive testing (NDT) of PCCP based on inspection zones, including acoustic, visual, and electromagnetic condition assessments. It is anticipated that all PCCP mains ≥ 36" diameter in the inventory will be inspected over a period of ten years.

A survey level inspection, as depicted in Figure 3, is proposed for small segments of PCCP in the inventory as well as for a selection of the longer PCCP transmission mains. Small segments are typically sections of PCCP less than 1,000 feet in length, that were installed as a repair or replacement to accommodate nearby construction projects.

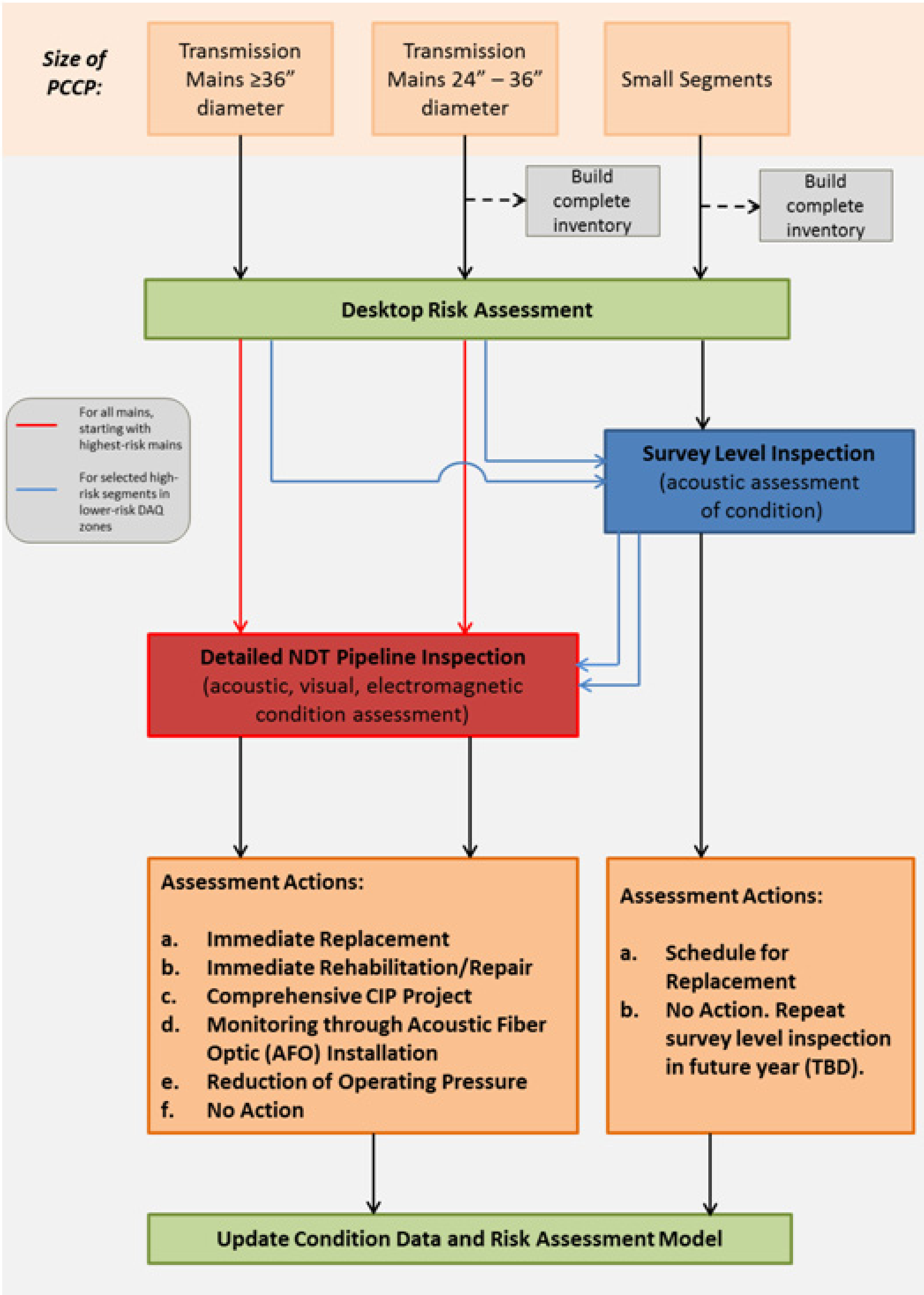

Figure 3. Overview of strategy for PCCP management in the Baltimore Metropolitan Water System.

The survey level inspection assesses pipe condition non-invasively based on the average structural modulus of the section and is significantly more affordable than comprehensive NDT inspection. Since detailed NDT inspection of all the small segments of PCCP in the system is cost-prohibitive, a survey level inspection of small segments can enable the City to manage these small segments of PCCP more effectively and repair and replace based on condition. The survey level inspection could also be used for a selected portion of PCCP transmission mains that are considered higher-risk segments in lower-risk inspection zones. The survey level inspection would reveal whether the prioritization of these segments for detailed NDT inspection or immediate repair and replacement would need to be accelerated.

Based on the results of the survey level inspection and detailed NDT inspection, the City will determine the course of action for each pipeline, as shown in Figure 3, in order to reduce the total likelihood of failure of the pipe. Based on industry experience, it is estimated that about one percent of the total pipeline inspected during the first assessment cycle will need immediate repair or replacement.

After inspection, the risk assessment model will be updated to reflect the new condition results for each pipe segment.

RISK ASSESSMENT MODEL

One of the first and most important steps in the PCCP program was to develop a detailed and comprehensive desktop risk assessment model. The model is being used to prioritize pipes for inspection, thus maximizing the inspection of high-risk pipes in the program's early stages and utilizing City resources effectively.

The desktop risk assessment model identified various factors for the likelihood of failure (condition) and the consequence of such failure (criticality). The likelihood of structural failure is a combination of factors that focus on the physical characteristics of the pipe and its operating conditions and installation, while the consequences of structural failure are measured through both direct and indirect impacts of a hypothetical failure.

Table 1 and Table 2 present the likelihood of failure and consequence of failure factors used in the risk assessment model.

Table 1. Likelihood of failure (condition) factors in PCCP risk model.

Item	Condition Factor	Definition
1	Wire class	Type of prestressed wire, related to the strength of wire – Class I, II, III, or IV
2	Cylinder thickness	Thickness of steel cylinder
3	Wire diameter	Diameter of prestressed wire
4	Severity rating (limit state status)	State of stress or strain in pipe under load from internal pressure and external soil and live load versus allowable limits
5	Pressure at zero compression (P_s/P_o) of the	Ratio of the working (service) pressure to the zero compression pressure of the concrete core

	pipe core	
6	Failure history	Number of recorded failures of pipe
7	Leak history	Number of recorded leaks in pipe
8	Data confidence score	Confidence in data based on data source – survey, as-built drawings, design drawings, GIS, or estimate
9	Installation year	Year the pipeline was installed
10	Time since last inspection	Period of time since last pipeline inspection, or presence of active AFO monitoring
11	Number of wire breaks	Total number of known wire breaks or unknown (not inspected)
12	Joint anomaly	Presence of joint anomaly, if inspected
13	Wire break zones	Number of quadrants in which wire breaks are present

Other likelihood of failure factors to be added to the risk assessment model once data is available include (1) pipe failure density (number of breaks per mile), (2) degradation rate, (3) uncertainty, (4) percent to yield based on the Finite Element Analysis curve, (5) leak identification, and (6) work order history (number of events per mile per year).

Table 2. Consequence of failure (criticality) factors in PCCP risk model.

Item	Criticality Factor	Definition
1	Pipe diameter	Size of pipe, between 16-in and 120-in in Baltimore
2	Redundancy	Measure of redundancy flow through pipe when failure occurs
3	Impact to critical infrastructure (buildings)	Measure of proximity to buildings where failure could cause damage or harm
4	Transportation impact	Intersection of pipe with primary or secondary roads using annual average weekday traffic data, where failures could cause major traffic disruptions or harm
5	Railroad impact	Measure of proximity to railways where failures could cause damage or disruptions
6	Impact to waterways, streams, and wetlands	Measure of proximity to waterways, streams, and wetlands
7	Impact to critical customers	Number of critical customers served by non-redundant pipes that would be impacted by pipe failure
8	Pressure	Pressure of pipe
9	Impact to gas lines	Proximity of pipe to gas line right-of-way

As the PCCP inspection program matures, the factors used in the risk assessment model may be revisited and modified.

PRIORITIZATION FOR INSPECTION

To inspect and monitor the inventory of PCCP in the Baltimore Metropolitan Water System in an efficient and cost-effective manner, DAQ (Data Acquisition) zones were established. A DAQ zone is a continuous pipe segment that can be monitored by a single acoustic fiber optic (AFO) DAQ unit. A single DAQ unit can monitor up to twelve miles of AFO; double DAQ units can monitor up to 24 miles, twelve miles in each direction.

DAQ zones will minimize the number of mobilizations required by field crews and make inspections more efficient. To minimize the number of DAQ units required to monitor the inventory of PCCP and maximize the reach of monitoring, strategic sites were selected to install the DAQ units.

Note that a DAQ zone can contain both high risk segments of PCCP and low risk segments of PCCP. It is anticipated that it will take approximately ten years to inspect all DAQ zones in the Baltimore Metropolitan System.

To implement the DAQ zone approach, the PCCP risk score was calculated across each DAQ zone, rather than for each pipe segment, using the pipe segment risk scores in the risk assessment model. Highest risk DAQ zones are inspected and monitored first; the proposed inspection schedule is shown in Figure 4. As previously stated, a condition survey of higher risk segments within a DAQ zone may be performed prior to the detailed condition inspection of the full DAQ zone to accelerate the condition inspection process if deemed necessary.

As baseline condition inspections are conducted across the inventory of PCCP, the risk scores in the risk assessment model will be re-analyzed using the new condition assessment results. The future inspection schedule may be adjusted as a result.

INSPECTION METHODS

Condition Survey: As part of the PCCP program strategy, a survey-level condition inspection may be conducted in advance of the more comprehensive condition inspection. This condition survey may be used for small segments of PCCP and a small selection of longer transmission main PCCP, such as higher-risk segments in the lower-risk DAQ zones, which would otherwise not undergo comprehensive condition inspection for several years.

The survey level inspection may include non-invasive acoustic technology to provide a composite view of the pipe condition by calculating the average stiffness of the PCCP section relative to the stiffness of other PCCP sections in the system. The City will further explore this technology and other case studies prior to application in the City's PCCP program. If used, the acoustic technology, which is less expensive than the comprehensive condition assessment technology currently used, will allow the

City to prioritize and advance repair or replacement of segments of pipe that yield unfavorable results.

Condition Inspection: PCCP inspection will be performed in each prioritized DAQ zone using a combination of acoustic, visual, and electromagnetic methods. The purpose of the inspection is to establish a baseline condition for every pipe segment and identify pipe segments in an advanced state of deterioration. The City intends to use a number of technologies in its toolbox to reduce time and costs of inspection.

The inspection methodology is largely dependent on whether the pipe can be taken out of service. Technologies that deploy free-swimming or tendered sensors will be utilized while the pipe is in service, while technologies that require the line to be depressurized or dewatered will be used when the pipe can be taken out of service without adversely impacting system operation. A combination of inspection methods will be employed to identify pipe segments that require repair or replacement, identify cracks and other signs of overloading, joint problems, and quantify the number of wire breaks.

Structural Analysis: If the inspection indicates a problem with a particular pipe segment, a non-linear, three-dimensional finite element analysis will be performed to relate the degree of deterioration to a corresponding risk of failure. The structural analysis models the current condition of the pipe, including any broken wires. A series of performance curves will be developed and used to build recommendations for the pipe segment being analyzed. An accurate structural analysis relies on knowledge of current operating conditions, such as earth cover and internal pressures.

REPAIR METHODS

The PCCP inspections will identify pipe segments in need of immediate repair or replacement, as well as segments that have minor damage but have remaining useful life and thus do not require immediate repair. The City is in the process of acquiring the services of on-call contractors to perform needed repairs on PCCP and design and repair PCCP using a carbon fiber composite system. For PCCP that cannot be dewatered, the City will have on-call contractors with the capability to install external post-tensioning tendon system repairs.

Rehabilitation Using Carbon Fiber Composite System: Depending on the degree of deterioration, PCCP may be repaired by reinforcing the pipe interior with a Carbon Fiber Composite System (CFCS). CFCSs have been utilized to line PCCP for decades and have the advantage of being a trenchless solution. CFCSs consist of a carbon fiber fabric impregnated with a resin or polymer. Once cured, the CFCS provides a structural strengthening system with the ability to carry the hoop tension of the failing host pipe.

Rehabilitation Using External Post-Tensioning Tendon (EPT) System: If the pipe cannot be taken out of service, the pipe can potentially be repaired externally using the EPT System. This system is composed of steel strands inserted into plastic ducts that are wrapped around the pipe exterior and tensioned to provide structural pipe support. The advantage of this type of repair is a reduction in time needed to perform the repair; the disadvantage is that the repair cost is typically higher.

Rehabilitation Using Structural Coatings: An alternative method for repairing smaller defects is the use of structural coatings, such as polyurea coatings. The coatings typically provide strong corrosion resistance and can have short curing times.

Replacement of Pipe Segments: This is typically the preferred approach when there is easy access to excavate, minimal traffic impacts, few nearby utilities to work around, and the pipe segment material is in stock when needed.

PROGRAM IMPLEMENTATION

At the time of the development of this report, the City has finalized the risk assessment model, developed a ten-year inspection schedule based on the results of the model, procured the services of inspection contractors, and planned for inspections to commence in late summer 2015. It is worth noting that the City, in this year, has conducted inspections of two high priority mains in the PCCP inventory under separate projects. These inspections have proven helpful to the City to understand challenges prior to the launch of the formal program, primarily the large amount of planning and coordination required among the many stakeholders.

Figure 4 presents the proposed inspection plan across the initial ten-year cycle.

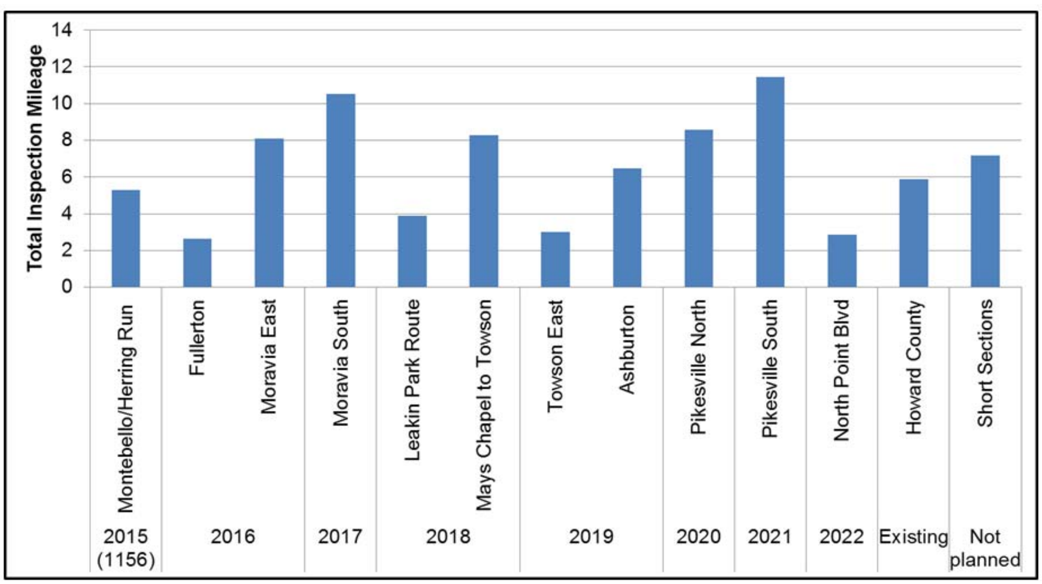

Figure 4. PCCP inspection plan by DAQ zone. *(Source: Pure Engineering Services, 2014.)*

Eleven DAQ zones have been identified for inspection. After inspections in each zone, the City will install acoustic fiber optic (AFO) monitoring systems to monitor the pipe segments for wire breaks. The AFO system uses fiber optic cable, a DAQ unit, and associated hardware and software to detect and locate prestressing wire failures while the pipe is in service. The progression of wire breakage can be used to assess the likelihood of failure through an analysis of the rate and density of wire breaks; the wire break data will also be added to the risk assessment model as they are collected.

Reports of recorded acoustic events associated with wire breaks will alert the City so remedial measures can be initiated. Pipes undergoing active distress can be taken out of service or rehabilitated before catastrophic failure occurs.

OUTREACH PLAN

Due to the significant investment the City is making to proactively assess and monitor PCCP, the implementation of the inspection program must proceed in parallel with a proactive and thorough outreach plan. Unlike other typical City programs such as the Sewer Lateral Inspection and Renewal Program, which address assets that impact each customer directly and visibly, the PCCP program addresses large transmission mains where, if the program is indeed successful, the customer would likely not feel any tangible impact. The program also has notable long-term operating costs associated with the permanent AFO monitoring. Thus, education to stakeholders on the importance of the PCCP program must be conveyed proactively and transparently.

The City intends to share the PCCP program plan with the Mayor, City Council members, customers, interest groups, and City neighborhood liaisons. The main messages conveyed to these stakeholders include the following:

- Risks associated with PCCP failure, including the history of breaks in Baltimore and other major cities
- The "failure avoidance" approach for the PCCP program, due to the high triple-bottom-line consequence costs
- The projected positive return on investment of the program
- Other benefits of the program to customers and the public, including:
    - Significantly reduced risk of critical water system failures, with improved reliability and safety
    - Limited rate impact for a reasonable investment
    - Balance of social, fiscal, and environmental benefits for stakeholders

DELIVERY CONSIDERATIONS

There were a number of delivery challenges that were experienced in the initial stages of the PCCP program or are anticipated as PCCP inspections roll out. These considerations include:

<u>Data limitations</u>: Baltimore City and County lack a complete inventory of the medium-size PCCP (20-inch to 30-inch in diameter), which hampers the ability to plan thoroughly for inspections or ensure thorough risk reduction. Additionally, the

existing records of concrete pipes in the GIS database do not match some existing PCCP records, requiring the City to manually marry the records to each pipe segment.

Coordination efforts: For each pipeline inspection and shutdown, a massive coordination effort is required between Baltimore City, Baltimore County, wholesale customers, inspection contractors, design and construction contractors, subconsultants, maintenance staff, plant operations staff, hydraulic modelers, and financial planners. For the two PCCP inspections the City has conducted in the past year, planning and executing the shutdown and inspection plan has taken longer than anticipated in the project schedule. This was due, in part, to the additional coordination required with multiple jurisdictions performing needed repairs during the transmission main shutdown. Coordination and schedule management will be a process of continuous improvement in future inspections.

Long-term monitoring costs: Once the City begins to install AFO systems, consistent funding will be required to monitor for wire breaks continuously. In this arena, City outreach to internal and external stakeholders is critical to ensure understanding of the importance of AFO monitoring even when budget and other resources are limited.

CONCLUSION

The City of Baltimore has made significant strides in the development and implementation of a comprehensive inspection, monitoring, and repair program for PCCP. It is hoped that the detailed upfront planning of the program – from the risk assessment model and strategic prioritization of inspections, to the procurement of capable contractors and proactive outreach – will contribute to a smooth and successful implementation of the PCCP program this year.

ACKNOWLEDGEMENTS

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Extending the Life of Existing Pipelines through the Use of a Retrofit Cathodic Protection and Internal Lining Program

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Abstract

This paper provides a presentation of a life extending system ("Anode Retrofit Program") for existing water mains by examining the planning, design and specification requirements. Included is a case study from the District of Columbia Water and Sewer Authority (DC Water) in Washington, DC. First, a discussion of the proper design and construction considerations for cathodic protection (CP) of water and wastewater transmission pipelines is basic to any asset management strategy. This includes the importance of understanding:

- Environmental conditions,
- Interior and exterior pipe conditions, and
- CP design considerations.

The second part of this presentation of an Anode Retrofit Program for cathodic protection of an existing steel main in Washington, DC. DC Water conducted an anode retrofit project as part of a water main repair on a 48-inch steel water main at the Brentwood Reservoir. The prime objective of corrosion control is to maintain a pipeline system free of corrosion at the lowest cost thereby extending the life of the asset. Washington, DC's corrosion control program have proven to be beneficial in the reduction of leaks while extending the useful life of the asset. In recent years the question regarding the use of linings or cathodic protection alone, or in combination, has been evaluated for the best economic choice, especially when viewed in terms of the life cycle of the asset. When corrosion protection is added to an existing pipeline asset that has a documented leak history, as part of an Anode Retrofit Program (ARP), it is possible to estimate the additional expected life. This provides decision makers with valuable data that can be used as part of a life cycle cost analysis for making informed decisions.

INTRODUCTION

Modern Cathodic Protection methods rely on more than galvanic or impressed current systems. These systems utilize coatings (exterior) and linings (interior) as part of a complete system for the successful protection of pipelines. However, what can be done to reduce exterior corrosion and protect existing pipes? How can CP systems be designed and installed to extend the life of existing pipelines? This paper provides a brief overview of the components of a complete Cathodic Protection (CP) System, what should be included when considering these systems and a brief presentation on an actual CP retrofit project.

PART 1 – DESIGN AND CONSTRUCTION CONSIDERATIONS

1.1 Corrosion Basics – Electrochemical Cells

Corrosion needs four components:

1. An anode
2. An electrolyte
3. A cathode
4. A return path

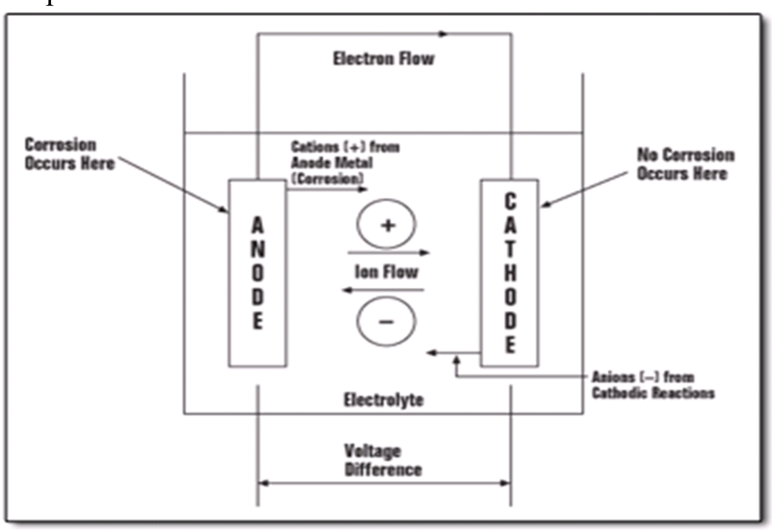

Figure 1: Electrochemical Cell

All forms of corrosion, with the exception of some types of high-temperature corrosion, occur through the action of the electrochemical cell (Figure 1). The elements that are common to all corrosion cells are an anode where oxidation and metal loss occur, a cathode where reduction and protective effects occur, metallic and electrolytic paths between the anode and cathode through which electronic and ionic current flows, and a potential difference that drives the cell. The driving potential may be the result of differences between the characteristics of dissimilar metals, surface conditions, and the environment, including chemical concentrations. There are specific mechanisms that cause each type of attack, different ways of measuring and predicting them, and various methods that can be used to control corrosion in each of its forms.

Corrosion prevention and control requires consideration of many factors before determining the specific problem and an effective solution, including but not limited to:

- Environmental conditions such as soil resistivity, humidity and exposure to salt water on various types of materials,
- Type of product to be processed, handled or transported,
- Required lifetime of structure or component,
- Proximity to corrosion-causing phenomena (e.g., stray current from rail systems, and
- Appropriate mitigation methods.

Cathodic Protection Systems

The objective with cathodic protection is to suppress the electrochemical reaction occurring at the anode. Under normal corrosive conditions, current flow from the anode results in a loss of metal at the anodic site with resultant protection of the metal at the cathodic site.

When a metal corrodes it takes up its own electrical potential known as the corrosion potential with respect to a fixed reference. When two dissimilar metals are connected in seawater, the metal with the lowest potential will suffer the greatest. In simple terms, the affinity of a metal to return to its natural stable state can be advantageously used in cathodic protection. Metals such as magnesium, zinc and aluminum have a greater desire to return to their natural state than mild steel. Connecting a steel pipe to for example, zinc, which will then become the anode and corrode in preference to the steel, can therefore control the corrosion rate of steel. In this example, the zinc anode is referred to as a sacrificial anode because it is slowly consumed (corrodes) during the protection process. It should be noted that if the mild steel has a lower potential than other connecting metals, e.g. stainless steel bolts, under the right conditions, the mild steel would corrode preferentially.

In general, the best method for extending the life of a pipeline is to install coatings and liners on the pipe along with cathodic protection for potential holidays in the coating. However, we as engineers are more often faced with existing pipelines in the ground that need to be protected.

Case Study: Washington, DC Brentwood Reservoir Pipeline Rehabilitation

The District of Columbia Water and Sewer Authority (DC Water) provides drinking water, sewage collection and sewage treatment to more than 600,000 residents, 16.6 million annual visitors and 700,000 employees in the District of Columbia Washington, D.C., USA. DC Water also provides wholesale wastewater treatment services to several adjoining municipalities in Maryland and Virginia. In addition, DC Water provides maintenance and repair of more than 250 miles of large diameter water mains in the District of Columbia.

DC Water has utilized an Anode Retrofit Program (ARP) on different types of water mains in recent years. For this presentation we will discuss the implementation of the ARP on two 48-inch steel water mains located at Brentwood Reservoir near New York Avenue in the North East quadrant of the city.

ARPs are becoming more attractive to utilities that include the installation of CP systems onto existing pipelines as part of their cleaning and lining program. These ARPs are resulting in life extensions of 20-years or more, based upon studies.

Brentwood Reservoir is a 25 MG underground reinforced concrete reservoir supplying the Low Service Area (172 ft. overflow elevation). Available record documents indicate that the steel inlet and outlet mains were installed under the reservoir construction contract in 1957-59. Under this contract, approximately 150 feet of 5/8” thick steel pipe was installed as inlet and outlet piping.

The inlet and outlet pipes to Brentwood Reservoir are separate, but conjoined within the valve/vent structure in an “H” configuration with valving to provide greater operational flexibility. The overflow discharges over a weir and then through a 48-inch steel outfall pipe. The drainpipe is 24-inch from the sump of the reservoir and through the wall into the valve/vent structure. The overflow and drain pipes connect together in the valve vault into a 48-inch steel overflow/drain pipe which extends to an outfall manhole approximately 140 ft. south of the valve/vent structure. At that location, the outfall exits through a flap valve before entering the storm drainage system. Refer to the Figure 2 for more details on the inlet/outlet-piping configuration.

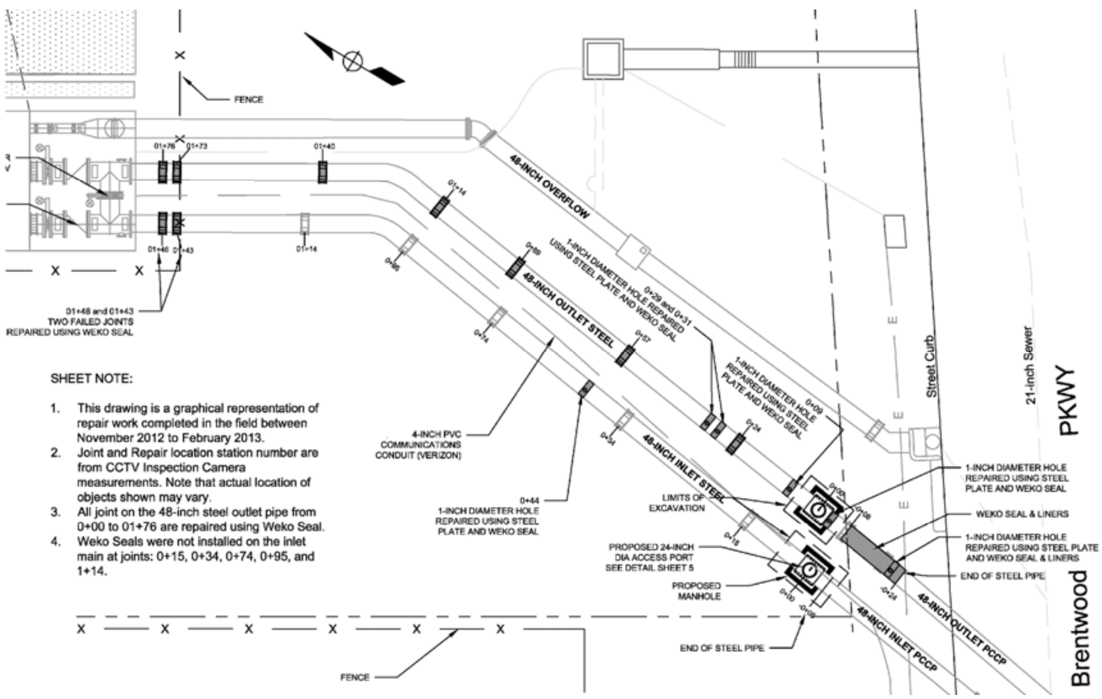

Figure 2. Inlet/Outlet Pipe Drawing

The steel inlet and outlet pipes connect at the street (Brentwood Pkwy.) to PCCP mains installed under a separate contract in 1959. There are mechanical couplings located just outside of the valve vault and there is a horizontal bend approximately 50 ft. south of the vault.

Prior to completing this ARP, a pipe condition assessment was conducted to ensure that a repair of the pipe was possible and the pipe’s life could be extended through the use of galvanic anodes. Upon completion of this investigation, a design for the system could be undertaken so that the pipe repair and CP system (ARP) install could be completed.

The 48-inch diameter steel inlet and outlet main was installed in 1957 with a coal tar enamel coating and lining according to AWWA Standard A203. When exposed the exterior coating was found to be in poor condition; the coating was cracked and was becoming disbonded from the pipe. The interior lining appeared to be in better condition except for one spool on the outlet main nearest the PCCP-Steel interface and random areas of minor corrosion within both pipes. Pits on the exposed portion of the pipe measured thickness losses of about 70%.

To be effective a CP system requires that the pipeline it is protecting is electrically continuous. The existing steel pipe used Dresser Coupling (see Figure 3) at the joints that can act as electrical insulators for pipeline. Without continuity an ARP requires bonds be installed on the pipe to connect the pipe sections electrically for better performance by the CP system. Additionally, the steel main was coated with a Coal Tar Enamel (CTE) to prevent corrosion.

Figure 3 – Dresser Coupling (typical)

Steel mains are known for suffering from pitting corrosion, a type of corrosion that can be more problematic than uniform corrosion in that pitting is much more difficult to detect, predict and design against. Pitting is a result of cavities or holes being produced in the material due to stray currents acting on the pipe.

It is difficult to detect due to the random nature of the pits that require a bit of luck to find when the investigators are excavating the pipe. Additionally, corrosion products can often cover the pits. A small, narrow pit with minimal overall metal loss can lead to the failure of an entire engineering system. Pitting corrosion is almost always a common denominator of all types of localized corrosion attack and the pits may assume different shapes.

In the case of a CTE coated steel pipeline, the pits will form where there are holidays (‘holes’) in the coating or where the coating has become disbanded from the metal. Disbondment can occur where the coating was poorly installed, or due to age.

All coatings are vulnerable to ageing, caused by a variety of influences such as thermal stress (fluctuations in operational temperatures), mechanical stress (vibrations), and exposure to ambient conditions (wet/dry cycles, freeze/thaw cycles).

The effects caused are changes in compositions and loss of essential properties. The result is that the metal is exposed to the environment and corrosion will then occur.

The cause of the leaks from these steel pipes could also be the result of a combination of several issues including dissimilar soils, bacteriological, stray current, etc. However, the most likely the leading cause is the failure of the pipe coating that has exposed the metal to the electrolyte (the soil). Due to the anodic – cathodic regions found by the engineer's testing galvanic pitting corrosion was occurring at various locations. Given its age, coating failure would continue to worsen and result in more pitting similar to that which has occurred at anodic locations along the pipe unless the pipe is protected cathodically.

Coat Tar Enamel (CTE) coatings from the 1950s were susceptible to oxidation and cracking, poor resistance to stress cracking, poor shear stress resistance, and a limited temperature range. This CTE coating appeared to be cracking, disbonding from the pipe allowing the steel to be exposed to the environment.

Based upon the engineer's field test data and the results of the pipe exposure, it is likely there were more pits on the outside of these mains that had not yet completely perforated the pipe. Without fully excavating the pipe to expose these corrosion pits the full extent of the damage could not have been known.

Based upon this analysis it was recommended that both the inlet and outlet mains should be cathodically protected to lower the potential of the metal to eliminate the anodic areas of the pipe. These protection measures would not repair existing damage but will halt the formation of new pits in the pipeline exterior.

Prior to the reservoir rehabilitation work, in the late 1990s a corrosion engineer conducted a field investigation of this pipeline. This investigation included pipe-to-soil potential surveys, stray current testing, soil sampling and analysis, and in-situ soil resistivity surveys over the pipelines. Based upon this and a previous investigation, the engineer concluded that the inlet and outlet water mains were sound. The engineers interpreted the data to indicate that the pipeline was in "good condition with no areas of corrosion", however, that opinion was premature given the lack of continuity of the pipe, the age of the coating, the low pH, the corrosive nature of the soil (soil box resistivity measurements indicated corrosive soils) and the stray currents (small but a shift >.800 Volts was recorded). (These results were determined through additional testing in 2012) Note that the better response to the data would be to have said that the pipe had a potential for pitting.

By 2012 the steel water main had already begun to leak and on October 25, 2012 DC Water's Water Program Management (WPM) team engineers were tasked with taking the lead role in identifying the best methodology to repair a leaking main at the Brentwood Reservoir. Additionally, the WPM team analyzed the repair versus replacement option that would consider a new design life of the pipeline.

A preliminary evaluation of the inlet main was conducted in October 2012 due to the appearance of a flow of water coming from an area above the inlet water main. The exact location of the leak was difficult to discern, as the inlet and outlet mains are parallel and about 10 ft. apart, center to center. Visual observations identified that the leak was surfacing in the upper third of the reservoir embankment indicating that the source of the leak was approximately 50 ft. from the curb.

Our team evaluated the feasibility of the following options for the repair of the inflow main:

1. Open cut excavation;
2. Manned entry via a new access MH installed near the transition of the PCCP pipe to steel pipe;
3. Manned entry via removal of valves and fittings in the pipe gallery at Brentwood Reservoir;
4. Manned entry utilizing both access points outlined in 2 and 3 above;
5. Video inspection.

Review of each alternative included consideration of reservoir down time and the effectiveness of each alternative in identifying the leak and providing a repair. According to the Department of Water Services (DWS) schedule (least amount of downtime of the reservoir) was of primary importance. The repair schedule of this main was complicated by the fact that the Crosstown water main was planned to be out of service until May 2013 making the Brentwood Reservoir critical to system operations. As such, down time of the reservoir had to be minimized.

After reviewing the options, manned entry via a new access manhole in the shallow area of the pipe, near the Prestressed Concrete Cylinder Pipe (PCCP)-Steel transition with video inspection was selected.

Based upon an analysis of the water main, it was determined that the best option would be to line the pipe, repair pit holes and install an anode retrofit system. This was considered as the pipeline was steel with Dresser style couplings and therefore suffering from pitting that resulted in leaks. Based upon the WPM's recommendations, DC Water undertook a plan that included draining the pipe, installing a new manned entry, repairing the leaks, and installing an anode retrofit as a CP system.

These repair activities were conducted on the inlet pipe between December 26 and 31, 2012 and on the outlet pipe between January 28 and 30, 2013.

The reservoir and associated piping is located in an area that, based upon field tests, lies within a corrosive soil. Prior to the repairs, additional field-testing was conducted to determine soils characteristics, stray current potential and soil corrosivity. Review of the pipe-to-soil field data indicated that there might have been anodic and cathodic potentials along the pipe alignment. The presence of these anodic and cathodic regions indicated a strong possibility of a galvanic corrosion reaction occurring on the anodic sections of the steel main. These conditions are conducive to increased probability of pitting failure if there is inadequate coating on the pipe or if holidays in pipe coating have developed. Further, the joints at the dresser couplings could also be subject to corrosion that can lead to premature failure. Based on the approximate location of the leak and the pipe construction details it was determined that the leak could have been located at a joint in the pipe although corrosion of the pipe wall could not be ruled out.

An analysis of the available drawings and maps and previous testing indicated that the pipe sections were joined using Dresser couplings that would make the mains non-continuous. Continuity is necessary for cathodic protection of the mains, so the engineer conducted continuity testing that confirmed that the pipes were not continuous. This lack of continuity would require that each pipe spool (or section) would need to have its own set of anodes if it would be properly cathodically protected.

Under their existing DC Water Internal Reline & Rehabilitation (IR&R) contract Corinthian Contactors, Inc. (CCI) hired Miller Pipeline (Miller) to conduct pre- and post-work closed-circuit television (CCTV) videos of the pipeline and install WEKO-SEAL liners to repair the joints and any holes found. Corinthian installed the access manholes and provided support to Miller Pipeline for traffic control and access.

Prior to the initiation of the water main repairs, Miller Pipeline completed a closed-circuit television (CCTV) video record. The first video was taken in the inlet pipe. Upon completing the inlet pipe repairs, Miller completed a video of the outlet water main. Using CCTV, the inlet main's video was available at to be viewed in real time, which allowed the engineers to determine the extent of the pipe damage. The outlet video file was corrupted and not recoverable. However, since the engineers were on hand to view the real-time results of the CCTV inspection in Miller's on-site trailer and had documented several leaks in the outlet main. The CCTV inspection found that outlet pipe had one section of more severe internal pitting corrosion, which required more extensive repairs.

The scope of work for the repairs was developed based upon the pre-work investigation. Upon completion of the pre-work investigation, the contractor installed:

- Access Manholes (2) on each pipe,
- Continuity Bonds at each pipe joint,
- WEKO-SEAL at each hole and joint in the water mains,
- WEKO-SEAL Liners on the corroded section of the outlet water main, and
- CP Testing Stations (2) at each manhole.

The contract also included the installation of one access manhole for each pipe (total of two) and a CP test stations for each pipe (total of two). Note that the installation of a complete cathodic protection system was not part of the repair scope of work. The anodes and relevant connections still need to be installed under a future contract.

Upon completion of the leak repairs, Miller conducted CCTV of the internal repairs. The CCTV post work videos are available to be viewed.

The engineer conducted electrical continuity testing of both the inlet and outlet water mains after completion of the repairs. Both mains were found to be electrically continuous. Therefore cathodic protection of the mains can be accomplished with an anode bed installed near the mains. Note that as there is no isolation provided between the inlet and outlet mains in the valve vault, the mains are electrically continuous with each other as one system.

The main was put back into service on February 12, 2013 with no further leaks on the mains.

Initially as part of the repair work, test stations were installed at each access manhole and anodes installed as part of a separate, near term project to protect the steel mains.

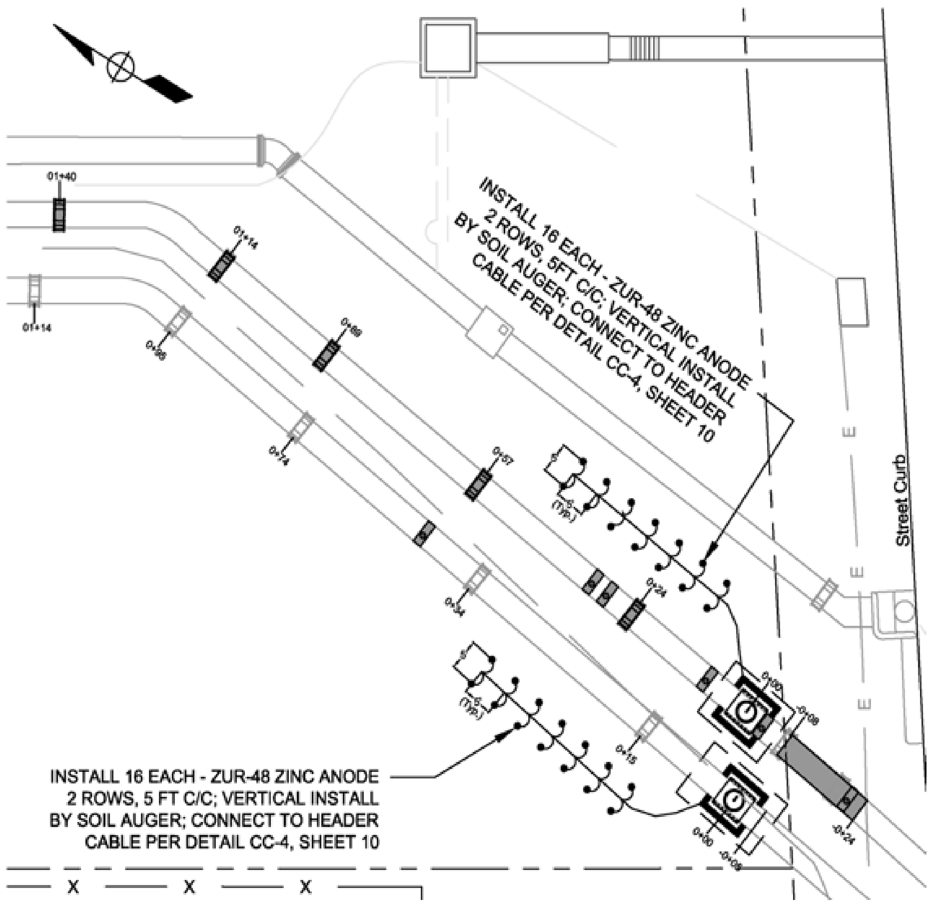

Figure 4. Anode Bed Design

For this piping system we recommended using a standard galvanic cathodic protection system with zinc anodes. This type system has the advantage of simple installation and retrofitting onto the existing steel mains. Additionally, maintenance is low, an external power source is not required and additional anodes may be added at any time to supplement or extend the existing system. See the Figure 4 for the proposed layout. The anodes would be designed for a >30 year life after which the mains would begin corroding once again unless new anodes were installed.

Note that magnesium anodes were not recommended due to the near connection to the PCCP that would be adversely affected by higher driving voltage of magnesium. Zinc has a lower driving voltage that means that more anodes would be necessary. PCCP wires can be subject to hydrogen embrittlement from the higher driving voltage from the magnesium anodes.

Costs

The project presented herein included installation of a liner system and had additional circumstances including the use of lump sum costing which made cost analysis difficult for this job. However, based upon projects completed on other sites and those reported by others (Schramuck, et al) the costs for installing a Retrofit Cathodic Protection system such as this ranges from approximately $10 to $20 per foot of pipe, depending on the location and job complexity.

Conclusion

Modern Cathodic Protection methods rely on more than galvanic or impressed current systems. These systems utilize coatings (exterior) and linings (interior) as part of a complete system for the successful protection of pipelines. This paper has attempted to provide an overview of the components of a complete Cathodic Protection (CP) System, what should be included when considering these systems and a brief presentation on an actual CP retrofit project. Anode Retrofit programs are now in place for many utilities and should be considered when deciding on pipeline replacement versus lining.

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Evaluating Remaining Strength of Thinning and Weakening Lined Cylinder PCCP Force Mains due to Hydrogen Sulfide Corrosion

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Abstract

Broken prestressing wire wraps are known as the primary cause of failure in a pressurized prestressed concrete cylinder pipe (PCCP) due to dissolution corrosion or hydrogen embrittlement. The effects of internal hydrogen sulfide attack to the concrete core can lead to delamination, spalls or pipe thinning and weakening of the concrete. The severity of the damaged concrete core typically increases from springline to the crown of the pipe. In some cases, it was observed that the interior concrete core at the crown of the pipe has completely deteriorated and the steel cylinder was exposed at the crown and corroded. For the case study, nonlinear finite-element analysis (FEA) is used to investigate the performance of a 30-inch and 48-inch Lined-Cylinder Prestressed Concrete Pipe (LCP) with a thinning and weakening concrete core. To understand the effects of concrete spalls and delamination, the stress and strain in the various components of a damaged LCP were investigated by applying the realistic loading while varying the number of broken prestressing wire wraps. The results were calculated for the aforementioned 48-inch diameter LCP design with 5, 35, 70, and 100 broken wire wraps and 30-inch diameter LCP design with 5, 25, 50, and 75 broken wire wraps. Based upon the results obtained, a comparison was performed between the effect of the broken wire wraps in the LCP with the deteriorated concrete core and a fully intact concrete core.

INTRODUCTION

PCCP was first manufactured in 1942 as lined cylinder pipe. The prestressing wire in lined cylinder pipe (LCP) is wrapped directly around the steel cylinder. A second type of PCCP was developed in 1952 in which the steel cylinder was encased in the concrete core. The typical diameter ranges for LCP and ECP are between 16 to 60-inches and 30 to 256-inches, respectively. Concrete core thickness of the ECP and

LCP change between 4 to 9 inches and 1 to 3 inches respectively, depending on the pipe diameter and loading.

Romer et al. (2007) and (2008) addressed major causes of failure of PCCP including ruptures or breakage in the prestressing wire wraps, leaking at the joints, cracks in the concrete core, hydrogen sulfide (H_2S, wastewater applications), low quality prestressing wire, overloading, and excessive surge pressures. Other researchers also discussed causes such as high chloride environment by Villalobos (1998). A 60-inch (152.4-cm) diameter PCCP was evaluated in terms of corrosion after 19 years of service in a high chloride environment. Chloride concentration of the mortar at the surface, middle, and at the wire surface was determined. The prestressing wires were found to be free of corrosion. The paper presents the results of the investigation and conclusions relative to the lack of corrosion on the prestressing wire. The effects of environment on the durability of PCCP were also evaluated by Price 1998. The quality of the mortar including lack of complete envelopment of the prestressing wires within the cement mortar coating was considered and concluded that the design or evaluation of prestressed pipelines must consider the effect of environment on an individual basis.

Rauniyar and Abolmaali (2013) performed two full scale experiments including three-edge-bearing tests to determine the behavior of ECP. They simulated the three-edge-bearing experimental test using three dimensional nonlinear analysis. They considered interactions between PCCP components and manufacturing process. Therefore, in their simulations the effects of shrinkage, creep and relaxation were considered.

Alavinasab and Hajali used a nonlinear finite element method to compare the structural integrity of a damaged PCCP when wire wrap breaks occur at the joint. They showed that the strength of the damaged pipe is not only related to the number of broken prestressing wire wraps but also to the location of the break regions along the length of the pipe. Based upon the obtained results, a comparison between wire break wrap in the middle of the pipe and the joint are presented and discussed. The results indicate the strength reduction at a joint for a low to medium number of wire wrap breaks was about 20%. However, if wire wrap breaks occur at the joint, it is anticipated that cracking in the pipe will occur much sooner than if the breaks occurred in the middle of the pipe. Also, Hajali and Alavinasab (2014) validated their computational model with experimental results obtained by the American Concrete Pressure Pipe Association (ACPPA) and conducted on three full-scale PCCP samples. The comparison was performed between the strain in the mortar coating, strain in the prestressing wires, and vertical displacement of the computational model and the experimental results. The comparison results showed a close agreement and the relative error for measured strains varied between 0.38% and 11.3% and for pipe deflection was between 5% and 9.5%.

Finite element analysis has been frequently used for modeling and evaluating the behavior of both types of PCCP. However, none of the research used in the literature have evaluated the effect of thinning and weakening the concrete core on the structural integrity of the PCCP. Thinning and weakening of concrete cores has been commonly seen in damaged force main LCPs and is important to determine the remaining useful life of these pipes and to prevent the uncontrolled release of wastewater into the environment. This study investigates the effects of thinning and weakening LCP force mains due to hydrogen sulfide corrosion using a three-dimensional nonlinear FEA.

FINITE ELEMENT ANALYSIS

The effect of concrete loss in LCP was modeled using a three dimensional nonlinear finite element analysis. Once the pipe was modeled correctly, all the loads including pipe weight, fluid weight, earth load, live loads, and internal pressure were applied. The FEA model predicts the performance of the damaged LCP with broken wire wraps and thinning and weakening of the concrete core utilizing a plasticity algorithm that simulates concrete crushing in compression regions. For this study, two LCPs with 30 and 48-inches diameter are modeled. The effect of spalls or pipe thinning and weakening of the concrete core being analyzed are illustrated in Figure 1(a). The concrete core thickness gradually decreases from springline to the crown of the pipe as shown in Figure 1(b). Different amounts of concrete loss at the crown were considered for our simulation. The maximum amount of concrete loss at the crown for the 30-Inches diameter LCP was considered 90% whereas 50% for the 48-Inch diameter LCP. The maximum amount of concrete loss at the crown of the pipe was based on our observation in the field inspections.

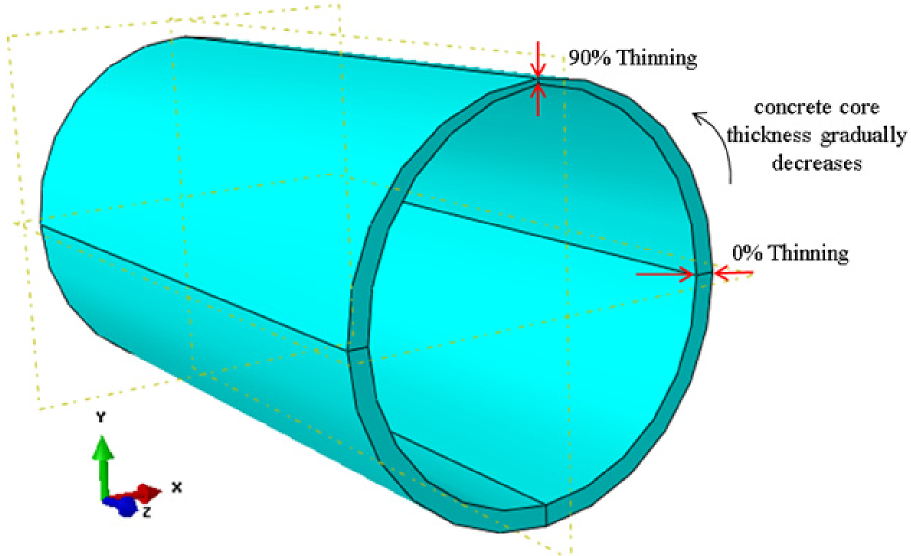

Figure 1 (a). Three-dimensional FEA model of the 30-inch diameter LCP with 90% concrete core thinning

More information about pipes geometry and components are given in Table 1.

Table 1. Pipe information

LCP Pipe Number	1		2	
Unit	**US**	**SI**	**US**	**SI**
Diameter of Pipe (inch,mm)	48	1219	30	762
Core Thickness (inch,mm)	3	76.2	1.88	47.75
Outside Diameter of Cylinder (inch,mm)	54.	1375	34	860
Cylinder Thickness (inch,mm)	0.06	1.519	0.045	1.14
Diameter of Wire (inch,mm)	0.192	4.877	0.162	4.115
Specified Coating Thickness (inch,mm)	0.875	22.225	0.813	20.638
Area of Steel Wire ($inch^2$,mm^2)	0.31	200	0.197	127.097
Wire Spacing (inch,mm)	1.12	28.47	1.26	31.9
Ultimate Strength of Wire (psi,kPa)	252,000	1,737,288	293,000	2,019,942
Gross Wrapping Stress of Wire (psi,kPa)	189,000	1,302,966	219,750	1,514,957

The prestressing wire used in Pipe 1 and Pipe 2 are 6-gage, Class III wire and 8-gage, Class IV wire respectively. It is assumed that the interface between the prestressing wires and concrete is perfect. A four-node quadratic shell element, in which each node has six degrees of freedom, is used in modeling the undamaged and damaged portions of the pipe. Figure 2 shows the 3-D mesh used in the analysis of the LCP. By virtue of symmetry, one quarter of a pipe section is modeled for the pipes with thinning and weakening of the concrete core.

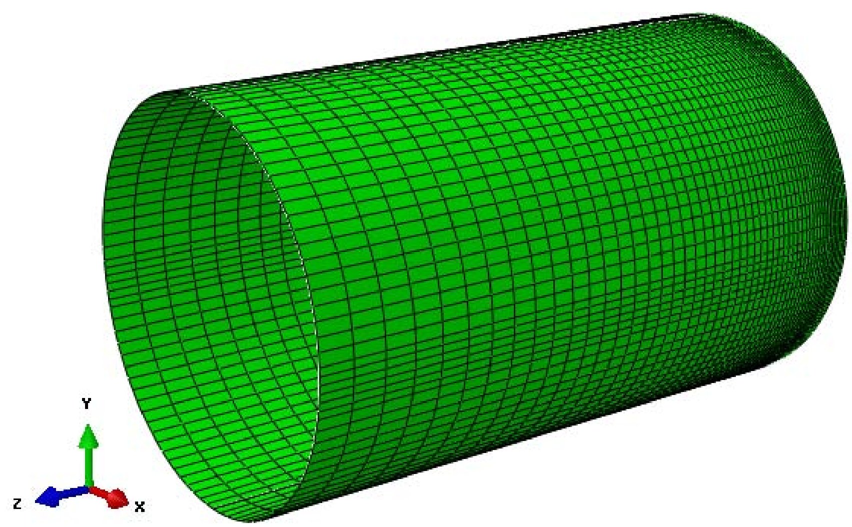

Figure 2. 3-D Mesh of the Damaged 48-inch Diameter LCP Model

The number of broken wire wraps was varied as shown in Table 2 in order to evaluate the serviceability and ultimate strength of the damaged pipes. The length of damaged section corresponds to the total number of wire breaks in the damaged section shown in Table 2.

Table 2. Damaged pipe length corresponding to number of wire breaks (WB)

LCP Class Designation	Damage Length, (inches)			
	5 WB	**35 WB**	**70 WB**	**100 WB**
Pipe 1	2.80	19.61	39.23	56.04
	5 WB	**25 WB**	**50 WB**	**75 WB**
Pipe 2	3.14	15.69	31.39	47.08

The concrete core portion of the LCP was modeled as solid three dimensional axi-symmetric elements. The nonlinear elastic behavior of concrete can be defined by the multi-linear stress-strain relationships governed by scalar damaged elasticity. The concrete ultimate compressive strength was modeled according to the AWWA C304. The LCP section with broken wire wraps was modeled based on the tensile strength of concrete and a plasticity algorithm that facilitated concrete crushing in compression regions. Cracking and crushing were determined by a failure surface, which formed the boundary between the undamaged zone and failure (damaged) zone. Once the failure surface was reached, cracking or crushing occurred. The model was subjected to loads corresponding to internal fluid pressure, pipe and fluid weights, and external earth loads as per the pipe dimensions. The internal pressure and amount of earth cover is shown in Table 3 for pipe No. 1, and 2.

Table 3. Damaged pipe length corresponding to number of wire breaks (WB)

LCP Class Designation	Working Pressure psi / kPa	Earth Cover ft / meter
Pipe 1	100 / 689.5	7.9 / 2.4
Pipe 2	30 / 206.8	9.8 / 3.0

MATERIAL PROPERTIES

The material properties used in the model are obtained from the AWWA C301 and C304 standard (AWWA, 2007). The modulus of elasticity of the core concrete was calculated from Equation 1 and modulus of elasticity of the mortar coating calculated from Equation 2. The Stress-Strain behavior of the concrete core and mortar coating are modeled based on the AWWA C304 Design Standard (AWWA C304, 2007).

$$E_c = (0.074)\gamma_c^{1.51}\left(f_c^{'}\right)^{0.3} \quad (1)$$

$$E_c = (0.074)\gamma_m^{1.51}\left(f_m^{'}\right)^{0.3} \quad (2)$$

where Υ_c is the concrete density, taken as 2320 kg/m^3 (145 lb/ft^3); f'_c is the 28-day compressive strength of concrete, taken as 37920 kPa (5,500 psi)); Υ_m is the mortar coating density, considered as 2240 kg/m^3 (140 lb/ft^3); and f'_m is the 28-day compressive strength of mortar coating, taken as 41368 kPa (6000 psi). The gross wrapping stress, f_{sg}, which is the stress in the prestressing wire during wrapping, is 75 percent of the specified minimum tensile strength of the wire, as shown in Equation 3. The yield strength of wire, f_{sy}, is 85 percent of the specified tensile strength of the prestressing wire, as shown in Equation 4.

$$f_{sg} = (0.75) f_{su} \tag{3}$$

$$f_{sy} = (0.85) f_{su} \tag{4}$$

The prestressing wire used is a 6-gage, Class III wire, with an ultimate strength, f_u, of 1737.5 MPa (252 ksi) for the LCP pipe No. 1. The Modulus of Elasticity of the wire, E_s, after wrapping at f_{sg}, for stress levels below f_{sg} is taken as 193053 MPa (28,000 ksi). The stress-strain relationship for the prestressing wire, after wrapping at f_{sg}, is given in Equation 5. The material property of prestressing wire is considered as shown in Figure 3 according to the AWWA C304.

$$f_s = \varepsilon_s E_s \quad \text{for} \quad \varepsilon_s \le f_{sg} / E_s$$

$$f_s = f_{su}\left(1 - \left[1 - 0.6133(\varepsilon_s E_s / f_{su})\right]^{2.25}\right) \qquad \text{for} \quad \varepsilon_s > f_{sg} / E_s \tag{5}$$

where ε_s is strain in prestressing wire.

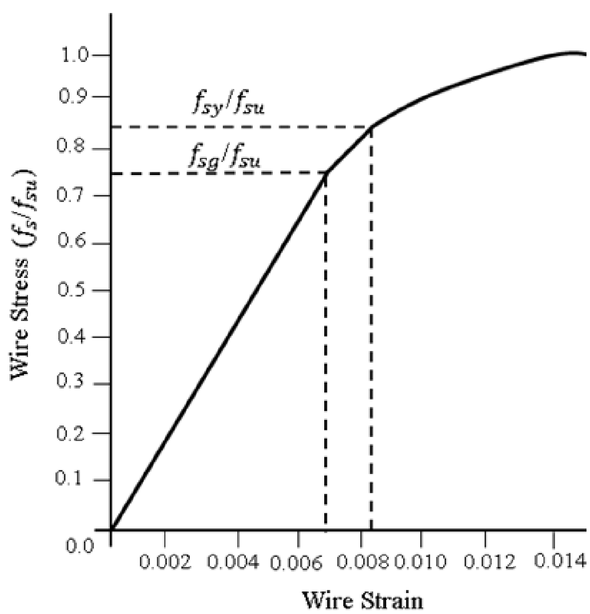

Figure 3. Stress-strain relationship considered for prestressing wires (AWWA C304, 2007)

RESULTS

In the damaged PCCP, the participation of concrete core as the load bearing component reduces significantly. This reduction would be more profound when we have concrete loss at the crown of the pipe as a result of hydrogen sulfide attack. In absence of concrete core, the majority of the load will be carried by the steel cylinder or transferred to the adjacent on damaged sections. Therefore, it is critical to monitor the level of stresses in the steel cylinder in the damaged section and PCCP component adjacent to the damaged section. Also, it is important to consider the effects of combined stresses to accurately predict the failure pressure. For this reason, we considered Von-Mises failure criteria. The Von-Mises criterion uses the effects of three dimensional stresses and compares them with the yield stress of the material. The von Mises criterion states that failure occurs when the energy of distortion reaches the same energy for yield/failure in uniaxial tension. Mathematically, this is expressed as,

$$\frac{1}{2}\left[(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2\right]\leq\sigma_y^2 \qquad (6)$$

For the structural evaluation, stresses and strains developed in the LCP were recorded and compared using the FEM computer modeling software, (ABAQUS). Evaluating the Von-Mises stresses in LCP would be one indicator for determining the remaining strength of the damaged pipe. Figure 4 and 5 show the level of stresses in in the prestressing wires and steel cylinder for the Pipe 1 with five (5) broken wire wraps, respectively at 1537 kPa (223 psi) internal pressure. This results shows how the stresses in the damage pipe distributed and transferred to its adjacent section. Also, Figure 6 shows the stress in the steel cylinder for Pipe 2 with twenty-five (25) broken wire wraps at 434 kPa (63 psi) internal pressure.

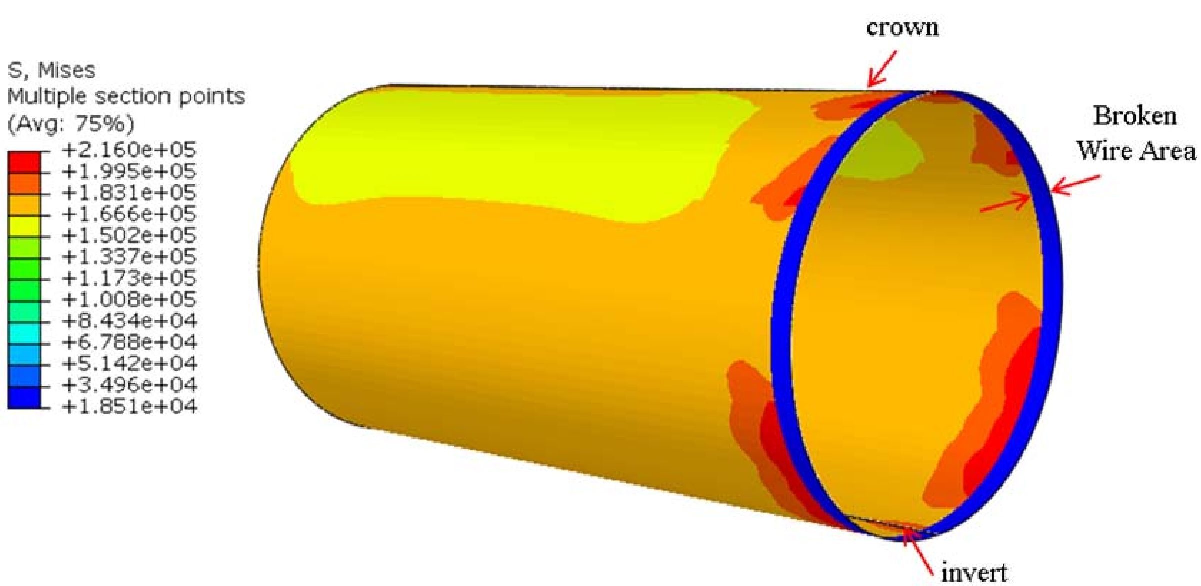

Figure 4. Stress* in in prestressing wires, 48-inch LCP, with five (5) Broken Wire Wraps

*stresses are in the graph were given imperial unit (psi)

Figure 4 shows, as an example, the stress developed in the prestressing wire for a PCCP with five (5) broken wire wraps at the barrel of the pipe. There is 50% concrete loss at the crown of the pipe and no concrete loss from the springline toward the invert for the 48-Inches diameter LCP. Symmetry was considered to model this pipe. Half of the pipe was modeled to reduce the computational analysis time. Von Mises stress is used since it is a good representative of the longitudinal or axial stress in the pipe (σ_L). The color gradient indicates the calculated range of stress for each element in the pipe model. Note that in Figures 4, 5, and 6 the stress is reported in imperial units. It is interesting to observe from the results that the highest amount of stress occurs near the location of the damage which in the figure 4 was shown with the red color. This is expected since the breakage of wire wraps in a particular region of the pipe will result in more stress concentration on the remaining undamaged wire wraps in the vicinity of the damage.

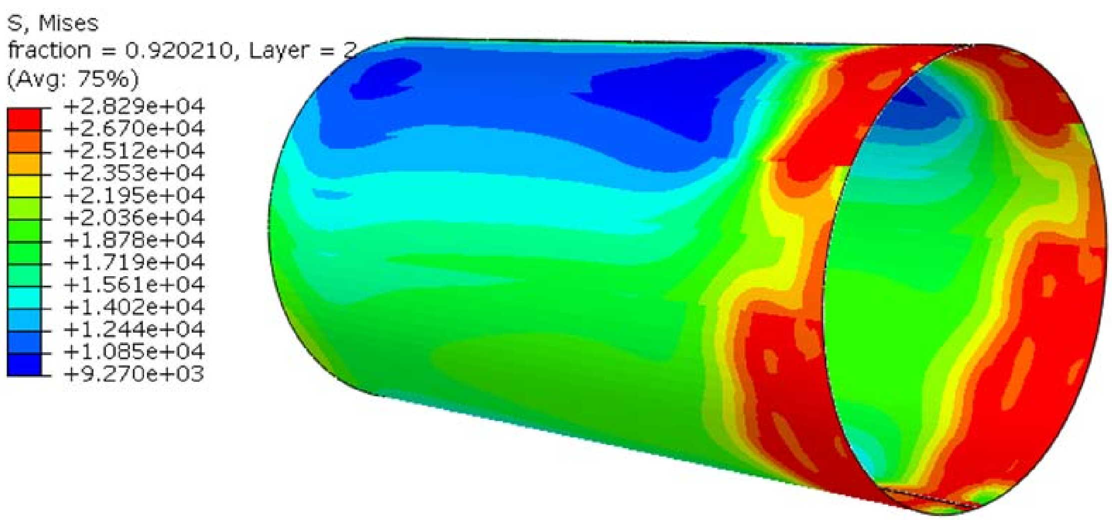

Figure 5. Stress* in steel cylinder, 48-inch LCP, with five (5) Broken Wire Wraps

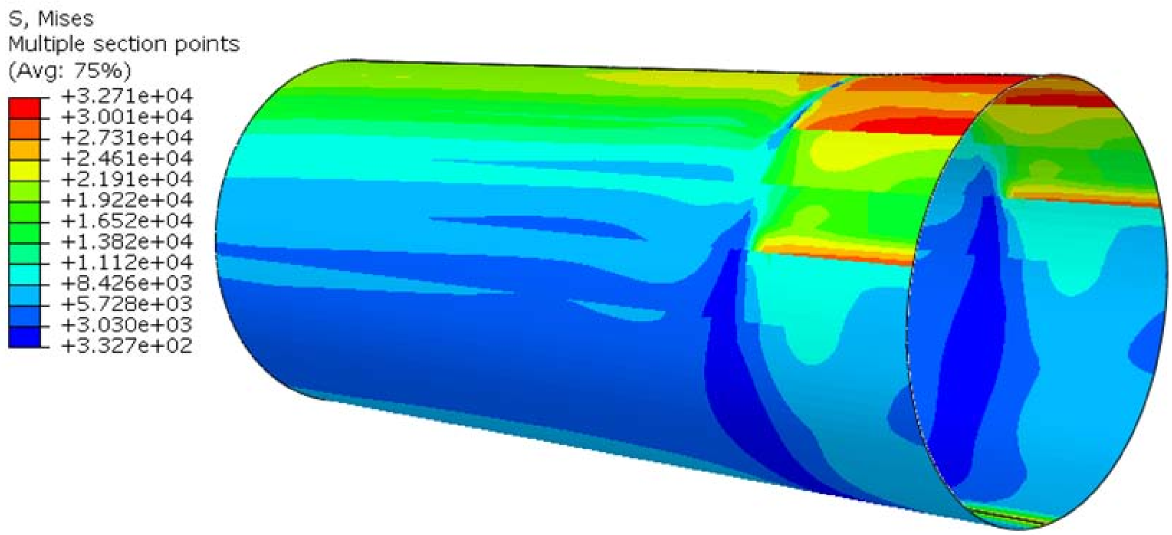

Figure 6. Stress* in steel cylinder, 30-inch LCP, with twenty-five (25) Broken Wire Wraps

The Yield and Ultimate Limits were evaluated similar to the limits defined in the AWWA C304 design standard. The corresponding yield and ultimate pressures in the damaged pipes were investigated by increasing the internal pressure of the pipe while the severity of damage was manipulated by increasing the number of broken prestressing wire wraps. Table 4 shows the pressures that cause the component of LCP reach to yield or ultimate strength with 5, 35, 70, and 100 broken wire wraps in Pipe 2. The amount of wall loss is the same for the pipes with different number of broken wire wraps. The critical pressures in which one of the pipe's components reaches to yield or strength limit for the Pipe 2 and Pipe 1 are given in Table 4 and 5 respectively.

Table 4. Yield Pressure and Ultimate Pressure in Pipe 2

Limit State	Pipe Condition	Number of Broken Wire Wraps			
		5	35	70	100
Yield Pressure	Intact Pipe	181	84	62	53
	Damaged Pipe	90	50	40	30
	Difference	50%	40%	35%	43%
Ultimate Pressure	Intact Pipe	264	127	100	95
	Damaged Pipe	120	63	53	43
	Difference	55%	50%	47%	55%

Table 5. Yield Pressure and Ultimate Pressure in Pipe 1

Limit State	Pipe Condition	Number of Broken Wire Wraps			
		5	25	50	75
Yield Pressure	Intact Pipe	174	75	55	45
	Damaged Pipe	150	60	40	30
	Difference	14%	20%	27%	34%
Ultimate Pressure	Intact Pipe	268	141	106	101
	Damaged Pipe	186	90	62	60
	Difference	30%	36%	42%	41%

The results showed that in addition to wire wrap breaks, thinning and weakening concrete core due to hydrogen sulfide corrosion decreases the structural integrity of the damaged pipe. The results of Pipe 2 indicate about 50% strength reduction as a result of 90% thinning and weakening of the concrete core at the crown. In Pipe 1, the amount of concrete loss at the crown was less than Pipe 1 (about 50%) and consequently the amount of strength reduction compared to the damaged pipe with no concrete loss was on average about 30%. Another observation by comparing the results was the level of strength reduction in the Pipe 1 was about the same for most of the number of broken wire wraps. However, in Pipe 2 the amount of strength reduction is higher for larger number of broken wire wraps. It can be concluded that the level of strength reduction would be stagnant for different length of distress sections when we have a significant the concrete thinning at the crown (90% loss). However, the results indicates that the level of strength reduction increase with increasing numbers of broken wire wraps when we have moderate or small concrete thinning (50% concrete core loss at the crown).

CONCLUSION

The effects of internal hydrogen sulfide attack to the concrete core in the force mains was observed as delamination, spalls or pipe thinning and weakening of the concrete. In this study, the severity of the damaged concrete core was increased from springline to the crown of the 30-inch LCP from zero to 90%. Another model was generated for 48-inch LCP and the maximum amount of concrete loss at the crown was considered 50%. The yield and failure pressure of the 30-inch and 48-inch LCP were evaluated and compared with the results for the same damaged pipe without concrete loss. Comparison of the results shows that for the 30-inch LCP there are about 35% to 55% reduction in the yield limit and ultimate pressure limit respectively. These values for 48-inch LCP were obtained in the range of 20%-40% additional strength reduction compared to the damaged pipes with intact concrete core.

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Pipelines at Bridge Crossings: Empirical-Based Seismic Vulnerability Index

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Abstract

Past and recent earthquakes have highlighted significant seismic vulnerability of utility lines, including water and wastewater pipelines, at bridge crossings. After the Canterbury Earthquakes (2010-2011), severe impact on utility lines at bridge crossings was reported. To identify the risk mitigation strategies for these systems, a performance based approach for the seismic assessment of integrated bridge-utility systems has been proposed. This paper focuses on findings from the first stream of the proposed framework, by presenting the product of the exhaustive data collection and collation, as seismic vulnerability indices. Firstly, an overview is presented highlighting performance of pipelines mounted on host bridges during the past earthquakes. Subsequently, the methodology employed to evaluate the seismic vulnerabilityindexforpipelines at bridge crossingsis presented. The methodology is based on data collation, which employs Geographic Information Systems (GIS) overlays of: the bridge and utility inventory in the Canterbury region; damage observations to bridge and utilities during the Canterbury Earthquakes (2010-11); and, the sustained ground shaking and ground deformation maps. The seismic vulnerability index will provide an initial basis for asset managers for pre-disaster screening and prioritization of existing potable water and waste water pipeline installations on bridge crossings, in seismic prone regions.

INTRODUCTION

The Canterbury Earthquake Sequence (2010-11) (CES) severely affected Christchurch city and its proximity. The CES was majorly affected by three major earthquakes: September 4 2010 Darfield Earthquake (M_w 7.1); February 22 2011 Christchurch Earthquake (M_w 6.3); and the June 13 2011 Earthquake (M_w 6.0), which were followed by several other aftershocks. The CES resulted in severe impact on the lifeline systems, with majority of the damage resulting from extensive liquefaction and lateral spreading, observed during all three earthquakes (Eidinger & Tang 2012).

The 2010 Darfield Earthquake (M_w 7.1) occurred at a depth of 10 km, 30 km southwest of Christchurch Central Business District (CBD), with ground shaking observed around 0.2g (PGA) in various regions and inducing high levels of liquefaction and lateral spreading. Whereas, the 2011 Christchurch Earthquake (M_w 6.3) occurred at a depth of 5 km, 10 km southeast of Christchurch CBD, inducing ground shaking of up to 1.0g (PGA) and 90 cm/s (PGV) in various regions; along with extensive levels of liquefaction and lateral spreading, of more than 500mm settlement and 400mm lateral offsets in some regions (Eidinger & Tang 2012, Cubrinovski et al. 2014).

Severe impact to utility lines mounted on host bridges was reported, during the CES. Even though, the structural performance of host bridges proved to be good with low observations of moderate to extensive damages (Palermo et al. 2010-11). The reported impacts to crossing utility lines include; leakage and breaks in water pipes in potable and sewage water pipes due to connection failure with the bridge and at pipe joints; and faults in power cables due to insufficient rotational capacity of cables at the bridge-embankment transitions (Eidinger & Tang 2012).

The need to develop a performance based approach against seismic risk for utility lines mounted on host bridges was highlighted through the damages reported after the CES. Moreover, the absence of seismic provisions for Bridge-Utility Systems (BUS) in the current national (NZTA Bridge manual 2013) and international codes (AASHTO 2009, Eurocode 8:part2 2005) of practice signifies the greater demand to address the need. Similarly, design guidelines for utilities crossing over bridges as developed by Bharil et al. (2001) and FEMA (1991, 1992) address the issue vaguely, without quantitatively defining the provisions.

Recently, a framework to develop performance based approach for the seismic assessment of integrated BUS has been proposed, to identify the risk mitigation strategies for these systems (Rais et al. 2015). The framework comprises of four streams, with each stream providing a unique output. The first stream is to collect and collate exhaustive data on bridge-utility systems in the Canterbury region, and detailed damage reports from the Canterbury Earthquakes 2010-11. The second stream involves simplified numerical analysis of the integrated BUS, developing the understanding of the basic seismic response of the system and the underlying uncertainties associated with it. The third stream focuses on detailed numerical analysis of the system, by incorporating results from FEM modelling of components integrated with Fluid Structure Interaction (FSI) effects into the global model of the integrated BUS, to develop fragility functions; highlighting the resilience of the system against ground shaking and liquefaction phenomenon. The final stream is to combine the fragility formulations and taxonomies, generated for the integrated BUS, with existing risk models to assess the functionality and socio-economic risk of the infrastructure networks, due to ground shaking and liquefaction susceptibility at bridge linkages. The intent would be to observe the interdependencies between infrastructure networks through these linkages, and to develop a risk scenario for Christchurch City as a retrofit prioritization or early warning tool for post-earthquake intervention.

This paper focuses on the preliminary findings from the first phase, after extensive collection and collation of the data, for the existing BUS in the Canterbury region and the damage observations from the CES. Firstly, an overview of the damage mechanisms observed during the past earthquakes is provided. Subsequently, the product of the exhaustive data collection and collation process is presented in terms of Seismic Vulnerability Indices (SVI). The methodology employed to evaluate the SVI is highlighted initially, which is followed by an overview of the characteristics of the Geographic Information System (GIS) overlays for the Canterbury bridges, potable water network and the hazard observations from the CES. The resulting correlations for SVI and the vulnerability estimation ranges are presented in the concluding part.

PAST PERFORMANCE OF BRIDGE-UTILITY SYSTEMS

There have been significant reports of damages to utility lines mounted on bridges, during the past earthquakes, highlighting their associated seismic vulnerability and the need to mitigate the issues. **A** compilation of various damages recorded to BUS are listed in Table 1, after going through reconnaissance reports of past earthquakes.

The major damages to BUS during the past earthquakes have been significant in areas prone to liquefaction or lateral spreading, as can be observed from Table 1. The primary reason is due to lateral spreading of embankments at river crossings, which tends to expose the upper part of the bridge abutment piles, or cause embankment settlement. Piles in the exposed region may undergo buckling which causes the abutments to rotate. Therefore, a utility line passing through the abutment would experience high stress concentration at the abutment-deck interface (Figure 1). Hence, leading to high curvatures induced in the utility line due to abutment rotation. Similarly, embankment settlement would induce high stress concentrations in the utility line at the embankment-abutment interface, where the embankment would impose high shearing stresses in the gravity direction.

Besides the dominant damage observed due to rotation of abutments, there have been instances where other failure modes were also observed. These include: failure of pipe at mid-span that may be possible due to ground shaking, as reported at the Durham Street Overbridge (CES) (University of Canterbury Bridge Damage Database (BDD)); buckling of pipe, as reported after the 1994 Northridge Earthquake (Schiff 1997); failure of primary and secondary connections that may be attributed to perturbations induced through FSI effects and ground shaking, etc., as reported at the Rokko Island bridge (1995 Kobe Earthquake, (Schiff 1998)) and at Bateman Avenue footbridge (CES) (BDD).

SEISMIC VULNERABILITY INDICES (SVI)

The vulnerability index method has been widely used around the globe, particularly in Italy, in the past few decades and is based on extensive data collection (Calvi et al. 2006). The vulnerability indices function as approximate indicators to portray the relationship between the seismic hazard and the seismic response of a given commodity. Where, the commodities can vary from buildings and bridges to infrastructure components and networks.

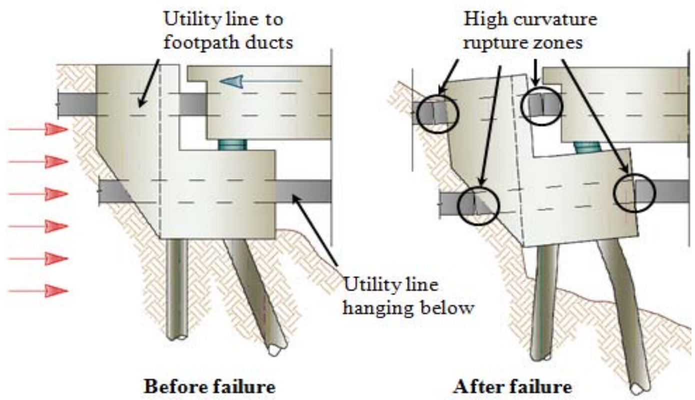

Figure 1. Failure mode of utility lines at abutment interface.

Table 1. BUS failures observed during past earthquakes.

Earthquake	Bridge	Observation	Comments
1976 Tangshan	800m bridge	Pressure pipe fail due to bridge collapse	FEMA (1992)
1979 Imperial Valley	Highway bridge	Pressure pipe deformed	Dobry et al. (1992)
1994 Northridge	Above ground pipes on saddles	Welded steel pipes, one cracked, one distorted	Schiff (1997)
1995 Kobe	Several dozen bridges	Pressure pipes damaged due to collapse of bridge; embankment settlement; support failure	Schiff (1998)
2008 Wenchuan	Several bridges	Pressure pipes damaged due to differential movement at abutments	Tang (2009)
2009 L'Aquila	Bridge in Onna	Pressure pipe failed due to embankment settlement	Tang & Cooper (2009)
2009 Padang	Kurao Bridge	Steel pressure pipe broke due to abutment damage	Tang (2013)
2010 Chile	Several bridges	Pressure pipes fail due to bridge collapse & displacement	Tang & Eidinger (2013)
2010-11 Canterbury	River crossing bridges	Pressure pipes broke, leaked or buckled due to embankment settlement; abutment damage; rigid supports; and deck displacements	University of Canterbury BDD, Palermo et al. (2012)
2011 Tohoku	Several bridges	Pressure pipes damaged due to bridge collapse or deformations	Tang & Edwards (2011)

Several models have been proposed for the estimation of vulnerability indices. For buildings, ATC-21 (ATC 1988) proposed to adopt the weighted sum of eleven principal parameters for estimation of vulnerability scores for masonry and reinforced concrete buildings. For bridges, similar models were proposed by ATC 6-2 (ATC 1983), Pezeshk et al. (1993), Maldonado et al. (2000), etc.; where, the methodology was based on the performance evaluation of the principal parameters, including construction era, superstructure type, bridge alignment, bearing type, pier type, span length, abutment type, foundation type, construction procedure, importance rating, etc. (Soberón et al. 2002). For other infrastructures, vulnerability indices for pipelines have been proposed by several authors based on comparison of the influencing parameters, such as pipe material, diameter, connection type, fault crossings, liquefaction susceptibility, etc. (Isoyama et al. 2000, Nojiima 2008, Zohra et al. 2012).

The general methodology of evaluating the SVI is based on weighing the influencing components and parameters of the system. Whereas, the influencing components for the BUS are the bridge, utility line and the connection between them; however, only the bridge and pipelines are considered for this study, due to data limitations. The influencing parameters for the bridge are identified as the bridge structural form, bridge material and the bridge construction era for this study. Similarly, for pipelines the pipe material and diameter are the main influencing parameters.

The SVI for a BUS is proposed here as a function of the relative ranking scores of its components and the hazard susceptibility. Where, the relative ranking score of the BUS components is a function of their influencing parameters. Hence, the SVI of the integrated BUS is proposed from the following relation:

$$SVI = (C_{BSF} + C_{BM} + C_{BCE}) \text{ x } (C_{PM} + C_{PD}) \text{ x } (C_H) \qquad \text{(Equation 1)}$$

Where, C_{BSF} is the score for the bridge structural form; C_{BM} is the score for the bridge material; C_{BCE} is the score for the bridge construction era; C_{PM} is the score for the pipeline material type; C_{PD} is the score for the pipeline diameter; and C_H is the score for the hazard susceptibility.

Isoyama et al. 2000 estimated the correction factors (‘*C’* values) by performing a combination of multivariate and regression analysis on the observed damage rates. In particular, regression analysis was used to estimate the correction factors for fragility relationships, to correlate the damage rate with the hazard intensity (Isoyama et al. 2000). In this study, the SVI is dependent on the hazard type, rather than on the hazard intensity, therefore, multivariate analysis is used to estimate the relative ranking scores (‘*C’* values). Weighted average of observed damage rates have been used to estimate the relative ranking scores for the influencing parameters of all BUS components. This is done by assigning weights to the damage rate, of each parameter under each hazard intensity category, to normalize the damage rate variation with the hazard intensity. These weights are averaged and then compared with the dominant parameter type to estimate the ranking score, against both ground shaking and the ground deformation hazards, for all influencing parameters. Whereas, the relative ranking scores for the hazard itself was obtained by comparison of damage severity scores for bridges against ground deformations (LRI) and ground shaking (PGA).

The following section provides an overview of the Christchurch dataset adopted for the evaluation of the component and parameter ranking scores, of BUS. The results of the coefficients in Equation 1 are presented in the concluding part.

SEISMIC VULNERABILITY INDEX (SVI) DATASET

Hazard. The use of GIS based ground shaking and liquefaction hazard maps for the two major earthquakes from the CES were employed for damage correlations, namely the 2010 Darfield Earthquake and the 2011 Christchurch Earthquake. As per availability of the damage data, the hazard maps (Figure 2) employed consist of: Peak Ground Acceleration (PGA) (USGS 2010, USGS 2011) for the two earthquakes; Peak Ground Velocity (PGV) (USGS 2011) for the 2011 Christchurch Earthquake; and the net Liquefaction Resistance Index (LRI) map (Cubrinovski et al. 2011) after both earthquakes.

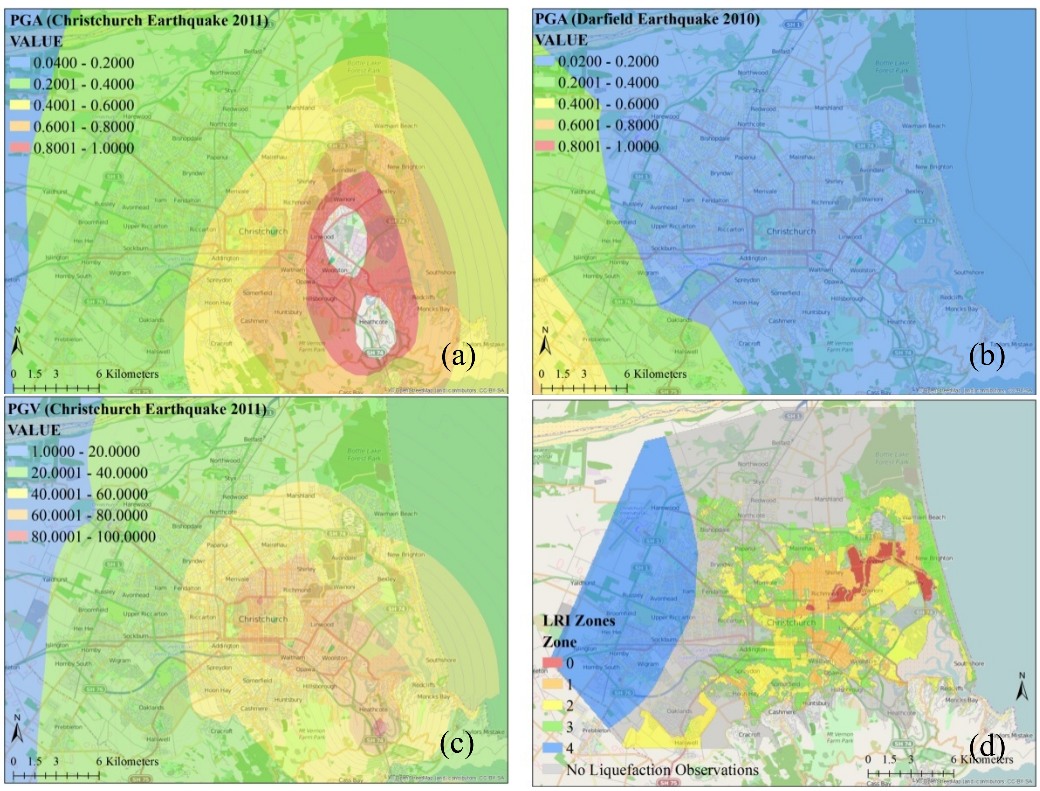

Figure 2. GIS Hazard maps for Christchurch: a) 2011 Christchurch Earthquake PGA map; b) 2010 Darfield Earthquake PGA map; c) 2011 Christchurch Earthquake PGV map; d) LRI map

The dataset in the BDD comprised of separate damage observations from both the 2010 Darfield and the 2011 Christchurch Earthquakes. Therefore, the datasets were correlated against the ground shaking intensity (PGA) maps from both earthquakes and the LRI map. Similarly, the potable water pipes damage dataset comprised of combined observations from both earthquakes; therefore, the dataset was correlated with the ground shaking intensity (PGV) map from the 2011 Christchurch earthquake and the LRI map.

Bridges. The sample dataset for bridges is adopted from the Bridge Damage Database (BDD) (Christchurch City Council 2011), developed under the supervision of Dr Alessandro Palermo. After the 2010 Darfield Earthquake, 800+ bridges were inspected and updated in the BDD; however, after the 2011 Christchurch Earthquake, since the damage was mainly confined in the Christchurch area, only 223 bridges were inspected and updated in the BDD (Brando 2012). The final dataset of 223 bridges from the BDD (Figure 3) has been adopted for this study.

Bridges in Christchurch are mainly short span bridges with median length up to 10 m, and with 86% of the bridges lying within the 30 m length (Brando 2012). The bridge structural form is composed of three main categories, including beam-deck bridges, arch bridges and culvert bridges. The bridge material composition is based on cast-in-situ concrete, precast concrete, steel, timber, and masonry bridges. Similarly, the construction era can be classified into two main categories, the pre-1960s and the post 1960s (Rais et al. 2015).

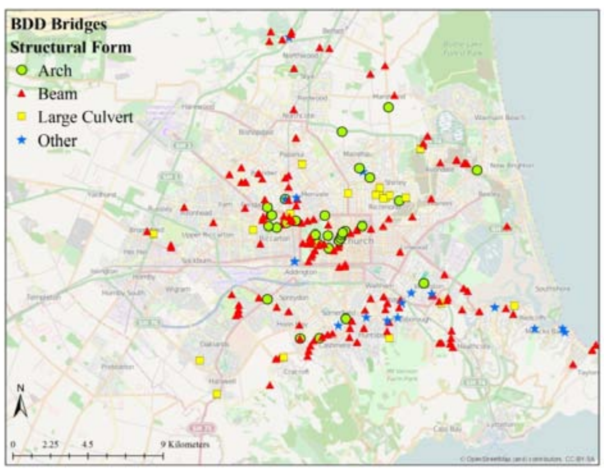

Figure 3. GIS layout of the 233 BDD bridges as per their structural form.

Major damage to bridges was observed in areas where severe liquefaction and lateral spreading occurred. However, the general performance of bridges was good, with low occurrence of severe damages. Severe damage was only observed in bridges spanning over the Avon and Heathcote rivers, where the lateral spreading effects were high. Damage modes included: settlement and lateral spreading of approaches; back rotation and cracking of abutments; and pier damage (Palermo et al. 2012).

The damage observations from the CES were used in this study to portray the seismic vulnerability of the bridge in BUS. The methodology used to quantify the damage severity of the BDD bridges was adopted from the study by Brando et al. (2012). Where, the damage observed to the BDD bridges during the CES were scored as per their severity to different components of the bridges, namely: deck and superstructure; bearings; piers; abutment; bridge pavement; approach pavement; approach settlement; services crossing the bridge; and the surrounds in the interaction zone with the bridge. These component damage severity scores were then collectively summed to portray the damage severity of the whole bridge (Brando et al. 2012). A similar approach was used for this study, where only the components that would interact with the mounted utility line were considered, namely: bridge deck and superstructure; bearings; piers; abutments and the approach settlement. Figure 4 shows the sum of the damage severity scores of the BDD bridges, as per structural forms, materials and construction eras, observed for different levels of ground shaking (PGA) and ground deformation (LRI Zones) parameters.

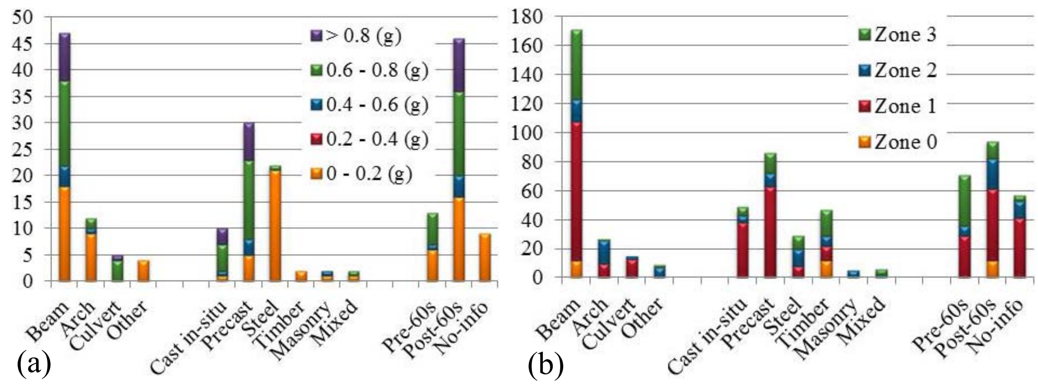

Figure 4. Damage severity scores for BDD bridges: a) Against observed ground shaking levels (PGA); b) Against observed ground deformation levels (LRI)

Pipelines. The potable water supply system of Christchurch (Figure 5) is based on a comprehensive underground pipeline network. The dominant material composition is comprised of High Density Poly-Ethylene (HDPE), Asbestos Cement (AC), Medium Density Poly-Ethylene (MDPE), Poly-Ethylene (PE), Poly-Vinyl Chloride (PVC), Cast Iron (CI), Galvanized Iron (GI), Modified PVC (MPVC), Un-plasticized PVC (UPVC), Concrete Lined Steel (CLS), Ductile Iron (DI) and Steel (S) pipes. Whereas, the diameters vary from 15 mm to 600 mm, depending on the transmission or distribution level of the pipe (Cubrinovski et al. 2014).

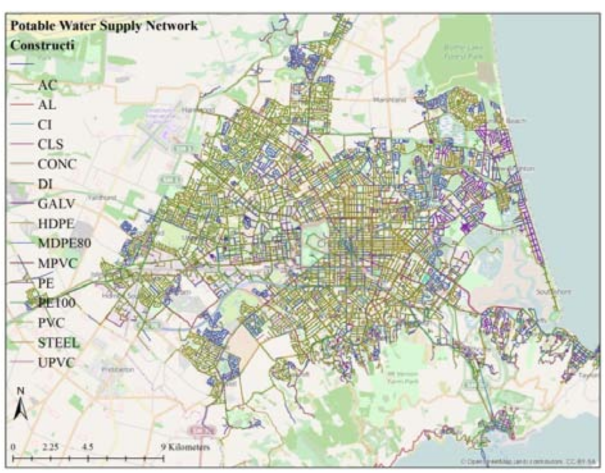

Figure 5. GIS layout of the potable water supply network of Christchurch as per material composition.

Several breaks were observed in the potable water network pipes during the CES, which led to disruption of water supply services. These damages include various failure modes, due to both liquefaction and ground shaking effects. A high number of failures were observed in high liquefaction zones, causing breaks in the pipes and its fittings (Cubrinovski et al. 2014). However, breaks were also observed in non-liquefaction areas, where the ground shaking effects were dominant and coupled with Fluid Structure Interaction (FSI) effects leading to breaks and leaks due to excessive pressure surges.

To quantify the damage observed to the potable water network pipes, during the CES, repair rates for the dominant pipe materials and diameters were evaluated against ground shaking (PGV) and ground deformation (LRI). Figure 6 shows the repair rates observed for different levels of PGA and LRI Zones.

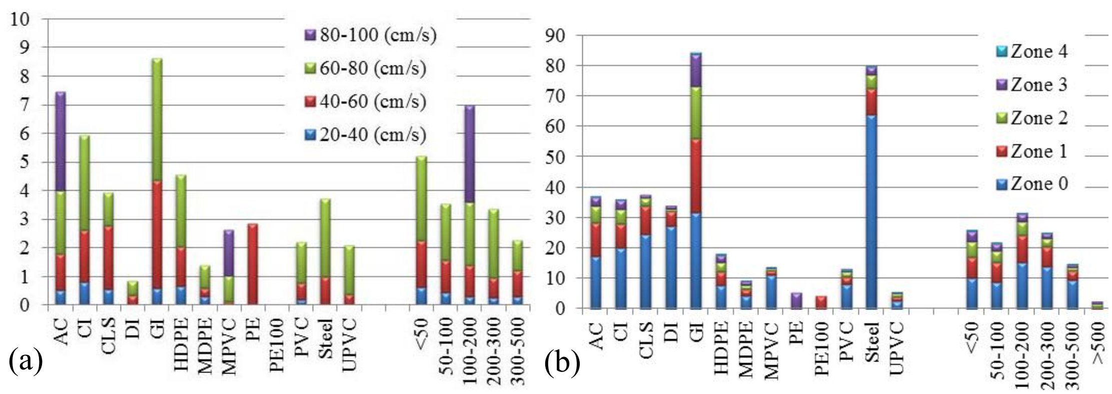

Figure 6. Repair rates (repairs/km) for potable water network pipelines: a) Against observed ground shaking levels (PGV); b) Against observed ground deformation levels (LRI Zones)

SVI CORRELATIONS AND RESULTS

The damage scores and damage rates, presented in the previous section for bridges and pipelines were correlated for each parameter type to obtain the relative ranking scores for the coefficients of Equation 1. The presented damage scores and damage rates were assigned weights as per their hazard intensity and averaged for each parameter type. The average damage score and damage rate was then normalized against the dominant parameter type, to obtain the relative ranking score for each parameter type. Figure 7 presents the relative ranking scores obtained for bridge and pipeline parameters. For the hazard coefficient, the average damage score for the bridge parameter types against ground shaking (PGA) and ground deformations (LRI) were compared with each other to obtain the relative ranking score (Figure 8).

The SVI ranges in between 2 and 47, with the proposed parameters. Alongside, vulnerability categories have been proposed to indicate the level of vulnerability for an integrated BUS combination. The vulnerability categories include: SVI < 15 (low vulnerability); 15 < SVI < 25 (moderate vulnerability); SVI > 25 (high vulnerability).

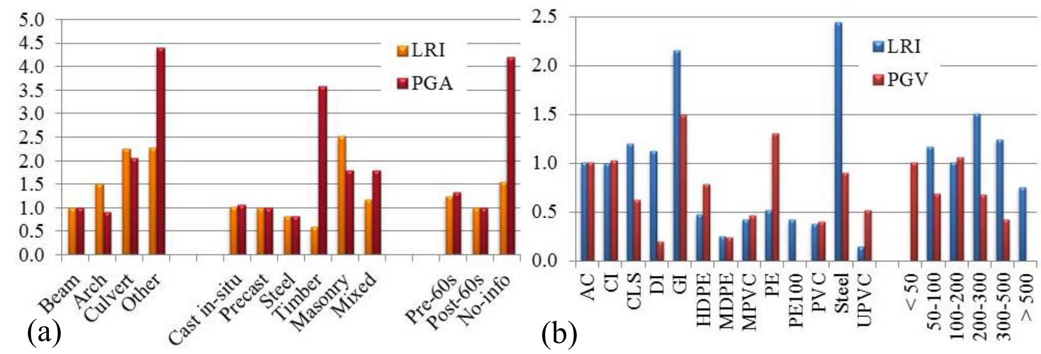

Figure 7. Relatives ranking scores for BUS components and influencing parameters: a) BDD bridges; b) Potable water pipelines

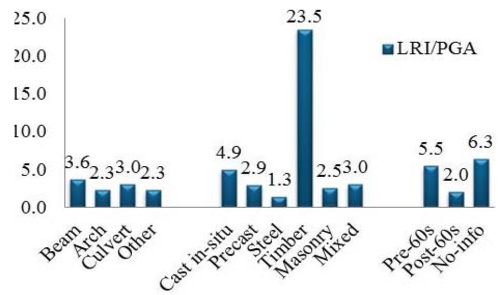

Figure 8. Relatives ranking scores for hazard

Table 2. Relative ranking scores for BUS components and parameters.

Factor	Bridge Type	Score	Factor	Pipelines Type[1]	Score	Factor	Hazard Type	Score
C_{BSF}	Beam	1.0	C_{PM}	AC	1.0	C_H	Ground shaking	1.0
	Arch	1.5		CI	1.0			
	Culvert	2.0		CLS	0.8			
	Other	2.3		DI	0.7		Ground deforma -tion[4]	2.8
C_{BM}	CSC[2]	1.0		GI	1.5			
	PCC[3]	1.0		HDPE	0.6			
	Steel	0.8		MDPE	0.3			
	Timber	0.6		MPVC	0.5			
	Masonry	2.5		PE	0.5			
	Mixed	1.5		PE100	0.4			
C_{BCE}	Pre-60s	1.2		PVC	0.4			
	Post-60s	1.0		Steel	1.5			
	No-info	1.6		UPVC	0.3			
			C_{PD}	< 50	1.0			
				50-100	0.8			
				100-200	1.1			
				200-300	1.1			
				300-500	0.8			
				>500	0.7			

[1] Al diameters are in mm; [2] CSC: Cast-in-Situ Concrete; [3] PCC: Precast Concrete; [4] Liquefaction and Lateral Spreading

CONCLUSION

The Darfield and Christchurch earthquakes of the Canterbury Earthquake Sequence 2010-11 (CES) proved to be devastating for the built infrastructure of Christchurch. Significant damage was observed to utility lines mounted on host bridges in Christchurch, mainly in high liquefaction or lateral spreading zones. Damage to BUS was also observed in low liquefaction areas, in high ground shaking regions.

A framework has been recently proposed to develop a performance based approach for the design and retrofit or new and existing bridge-utility systems. The preliminary findings of the first phase (extensive data collection and collation of BUS) are presented in this paper. Seismic Vulnerability Index (SVI) was proposed in this study to estimate the associated vulnerability in different BUS combinations. Alongside, the methodology employed to develop the SVI and the damage observations to the BUS components, during the CES, along with their primary influencing parameters were presented and discussed.

The SVI can be used as a preliminary indicator for identifying vulnerable links in the utility infrastructure networks, at BUS linkages. The early warning would enable the asset managers to develop prioritization strategies for retrofit or strengthening. Alongside, the SVI can be also be employed by asset managers to estimate the feasibility of mounting a utility line on an existing host bridge, by the assessment of the influencing parameters.

Further findings on the framework are expected to meet the time frames, as have been proposed in Rais et al. (2015). These include the non-linear seismic response of integrated BUS; seismic fragilities of the integrated BUS; detailed non linear seismic response of the BUS components; risk assessment of utility infrastructure networks at BUS linkages with reference to network functionality; mitigation measures for reducing the overall seismic risk at BUS linkages.

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Benefits of Global Standards on the Use of Optical Fiber Sensing Systems for the Impact of Construction of New Utilities and Tunnels on Existing Utilities

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Abstract

Distributed Optical Fiber Sensing is a mature technology given its strong record of over 20 years. Nevertheless, underground utilities are yet to embrace it as an everyday tool despite its enormous capability. One dimensional long buried utilities and tunnels offer the best application for the use of this technology. Research studies around the world offer the promise of this technology in monitoring the impact of ground movements on underground utilities and tunnels. No application standards existed that governed the use of this technology within any jurisdiction in the world in September 2012. A global task group on optical fiber sensing systems (OFSS) was born to become a unique pool of talent and experience on the subject with over 40 leading experts from 17 countries, which went on to author two companion standards American Society for Testing and Materials (ASTM) F3079-14 and F3092-14, within ASTM Technical Committee F36. This paper provides a brief overview of how OFSS work, what is in these standards, why OFSS is poised to become the most versatile innovation among all measurement tools for field monitoring, what problems the task group faced during the development of the standards and how the members of the task group resolved these problems, what the benefits are of such global standards and the future plans for the global OFSS task group. The most paramount goal of the authors is to share the lessons they learned during the development of the standards with the delegates of this conference.

INTRODUCTION

When the corresponding author served as a consultant to DC Clean Rivers Project tunnels, the amplitude of the ground movements measured was in fractions of an inch while the noise in the traditional instrumentation systems used was even higher, Jeyapalan et al. (2014, 2015). This led the team to consider OFSS methods, but ultimately they were not implemented. The lack of consensus standards for OFSS methods was a contributory consideration in the client's decision. This market need was the primary driver for the birth of the ASTM F36 Global OFSS Task group.

Some of the significant publications which demonstrated the advantages of using OFSS in civil infrastructure include Vorster et al (2005) on assessing the impact of ground movements due to construction activities nearby on buried pipelines, Briançon et al. (2004) and Nancey et al. (2007) on a composite fiber-optic sensor-enabled geotextile for soil strain assessment using the Fiber Bragg Grating technology, Calderon and Glisic (2012) and Glisic (2011) on field observations using OFSS and the accuracy of embedded long-gauge optical fiber strain sensors, Glisic and Inaudi (2008) on the use of OFSS for structural health monitoring, Klar et al (2008) on analysis and field monitoring, Mohamad et al. (2014) on temperature and strain sensing using Brillouin Optical Time Domain Reflectometry (BOTDR), and Mohamad et al. (2012) on tunnel induced response of old brick tunnels and new tunnels. Artières et al. (2010) showed also the use of OFSS to monitor hydraulic structures. Iten (2011) and Iten et al. (2011) demonstrated the effective use of optical fiber sensing systems to a wide range of geotechnical applications.

The effect of distributed Brillouin scattering is the most widely used form of OFSS technology, which provides a monitoring technique to measure strain and temperature along the optical fiber cable. When a light pulse travels through the optical fiber core, most of it is transmitted from one end to another (assuming no breakages or kinks in between) following the principle of total internal reflection while only a small fraction is back scattered in the direction of the source due to the tiny imperfections in the density of the core along the cable. Different components of the back-scattered light can be identified, including the Brillouin scattering components, such as the peaks shown in Figure 1; these are carefully analyzed and used to measure changes in temperature or strain along the fiber.

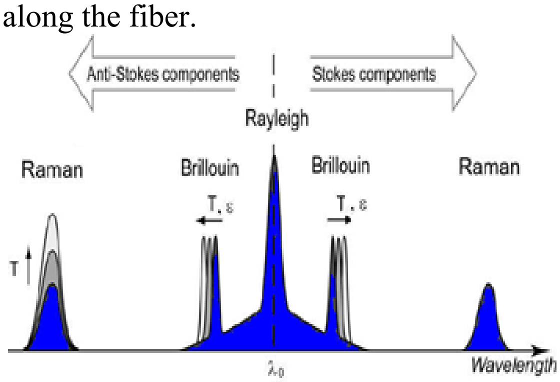

Figure 1. Brillouin peaks as functions of wavelength

Therefore, the optical fiber cable itself plays the role of an almost infinite number of strain and temperature sensors for long distances. Standard telecommunication optical fiber cables designed to protect the optical fibers from the surrounding environment can serve as temperature sensors. In a strain sensing cable, however, the surrounding medium must efficiently transfer the strain to the optical fiber core. That means any strain applied to the cable coating must be directly transferred to the fiber core, where the strain is measured by Brillouin backscatter. Many optical fiber sensing cable designs exist nowadays with different characteristics with some examples shown in Figures 2 and 3. Optical fiber cables embedded in geotextile to enhance transfer of soil movement to the fiber are shown in Figure 4.

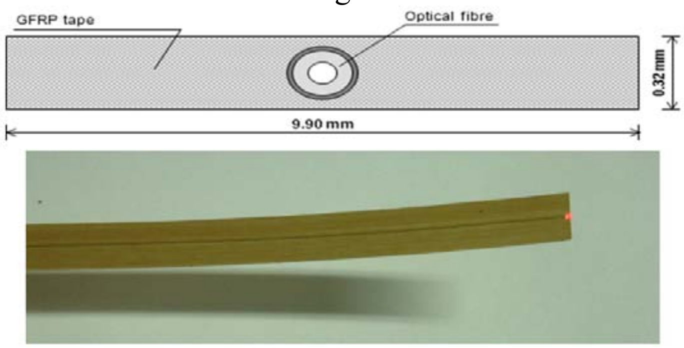

Figure 2. Components of an optical fiber strain sensing cable

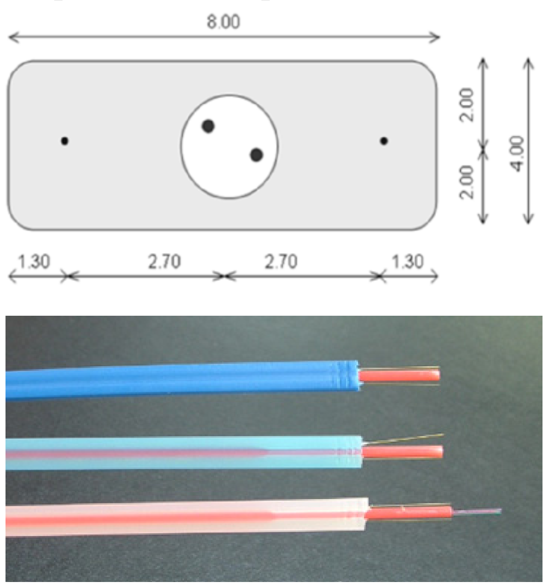

Figure 3. Parts of an optical fiber strain and temperature sensing cable

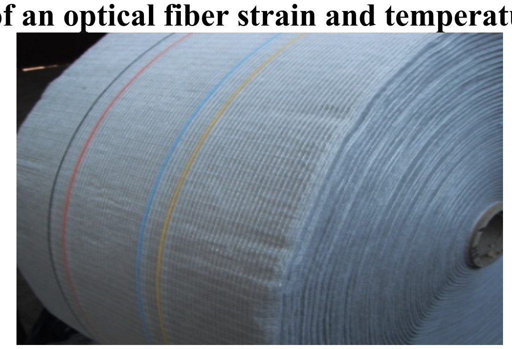

Figure 4. Sensor-enabled geotextile with strain and temperature cables

In the Brillouin Optical Time Domain Analysis (BOTDA) technology, two laser beams are injected into an optical fiber core from both its ends in Figure 1. One is called the pump signal, being a pulse-modulated (for BOTDA systems) or a sinusoidally modulated, for Brillouin Optical Frequency Domain Analysis (BOFDA) systems laser beam of a unique wave profile it is the continuous wave(CW) probe laser, sometimes referred to as the Stokes laser. The interaction of these two laser beams produces an acoustic wave called "electrostriction." The pump signal is backscattered by the phonons, and the energy is transferred between the pump signal and the CW probe light.

The Brillouin Loss Spectrum (BLS) or Brillouin Gain Spectrum (BGS), as the function of frequency difference between the two laser beams, is measured by scanning the frequency of the CW probe light. The value of the strain or the temperature can be estimated using the shift of the peak frequency of BLS/BGS (Brillouin frequency), whilst its position calculated from the light round-trip time as shown in Figure 5.

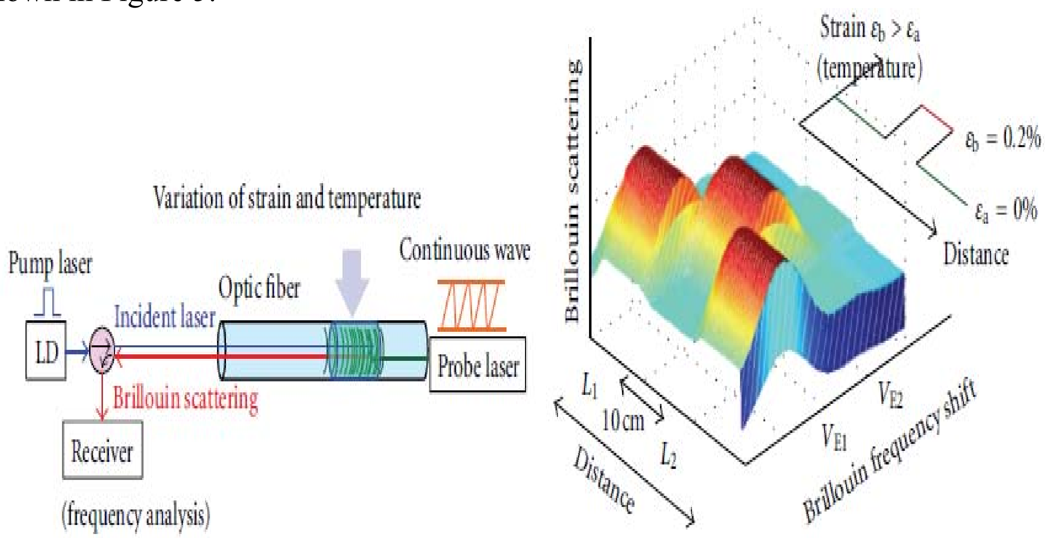

Figure 5. Principal components of the BOTDA system

Similar set up for the Brillouin Optical Time Domain Reflectometry (BOTDR) technology is shown in Figure 6 which requires sending the light pulse from one end of the core and hence negating the need for a closed loop. Therefore, an appropriate interrogator, with a graphic user interface, as shown in Figure 7, and the software, for example shown in Figure 8, can acquire and keep track of the position and the magnitude of the strain or temperature at hundreds of thousands of locations along the route of the optical fiber cable, essentially in almost real time. Results from such BOTDA and BOFDA systems are shown for strains in Figures 9 and 10, respectively.

EFFECT OF BRILLOUIN SCATTER FACILITATING TEMPERATURE AND STRAIN MEASUREMENTS

Brillouin scatter is extremely sensitive to any changes in temperature and strain experienced by the optical fiber;in this regard, most environmental stimuli the optical fiber is exposed to can be correlated to temperature and strain, and measurements can be made on the effects of such environmental stimuli on the serviceability of a buried pipeline or the ground responding to the impact of tunneling or new utility construction. The frequency shift, ν_B, can be calculated using:

$$\nu_B = \{2nVa\}/\lambda_o \qquad (1)$$

in which, n is the effective refractive index of the propagating mode, Va is the acoustic wave velocity in the optical fiber and λ_o is the vacuum wavelength of the incident light. The Brillouin frequency shift is affected by the acoustic wave velocity, which can be expressed for homogenous, isotropic, linearly elastic solids as

$$Va = \{ K /\rho \}^{0.5} \qquad (2)$$

where, K is the bulk modulus and ρ is the density of the optical fiber, respectively. The density of the optical fiber is dependent on temperature; therefore, the Brillouin peak shifts can be plotted as a function of the difference in the frequency between the laser pump and the signal varying with temperature. Similarly, any deformation or strain in the sensing fiber can be tracked. In summary, the temperature and the strain induced in the optical fiber can be measured using the effects of Brillouin scattering.

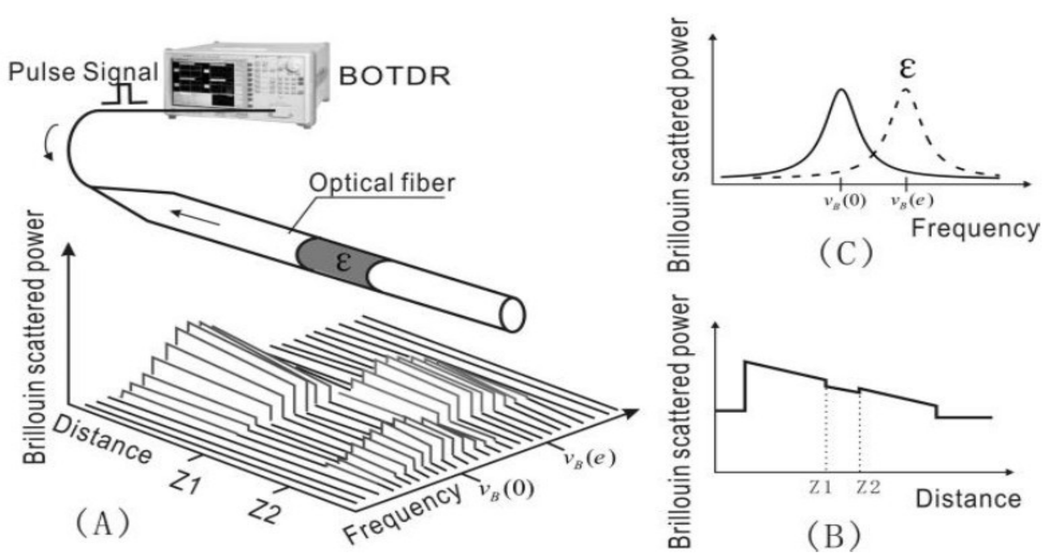

Figure 6. Details on a BOTDR system

SCOPE OF THE STANDARDS F3079 AND F3092

The F3079 Standard specifically addresses the standard practice for the use of distributed optical fiber sensor systems (DOFSS) for monitoring ground movements during tunnel and utility construction and its impact on existing utilities. It applies to the process of selecting suitable materials, design, installation, data collection, data processing and reporting of results. This standard practice applies to all utilities that transport water, sewage, oil, gas, chemicals, electric power, communications and mass media content. This practice applies to all tunnels that transport and/or store water or sewage and to tunnels for hydropower, traffic, rail, freight, capsule transport, and those used for dry storage. The second standard F3092 is companion to the first in that it includes more than 400 terms commonly used in optical fiber sensing systems, utilities and tunnels.

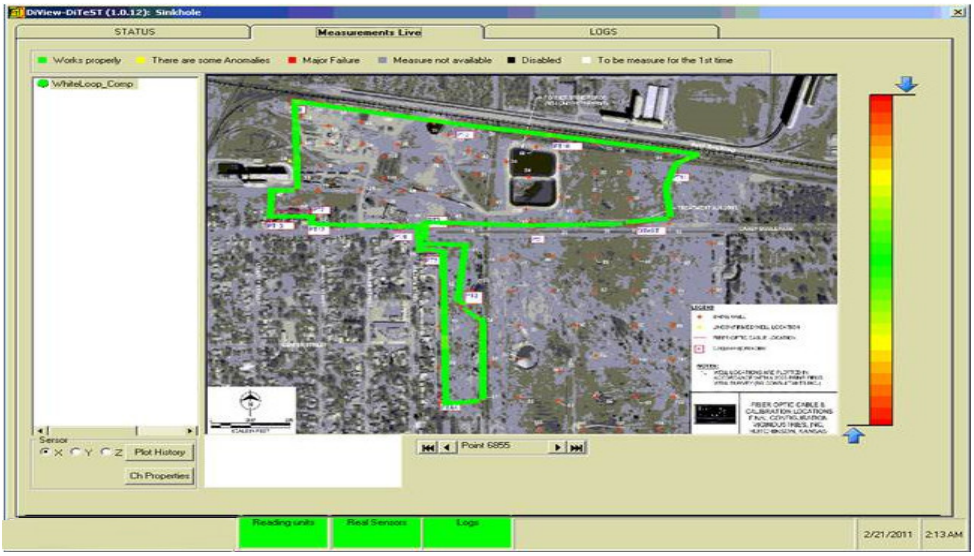

Figure 7. Typical graphic user interface

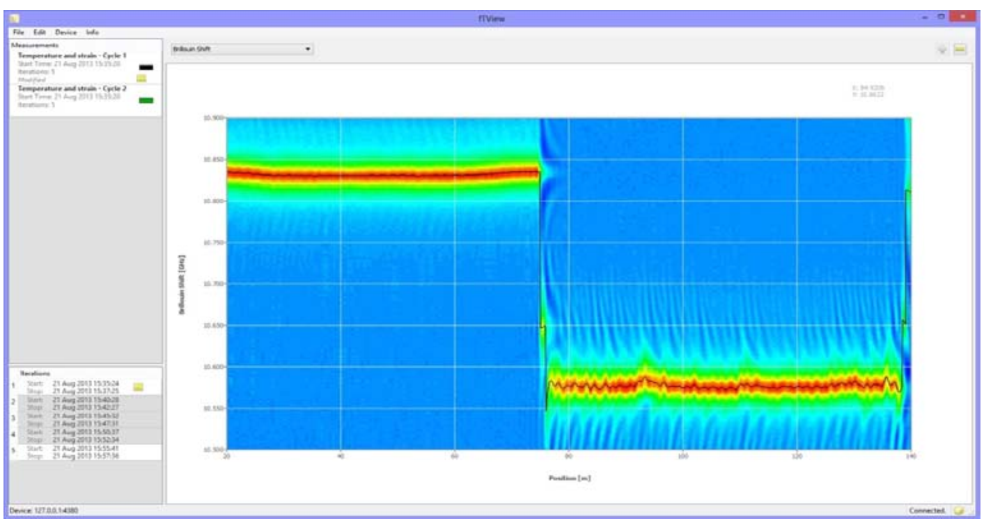

Figure 8. Typical screen shot of software of BOFDA

SIGNIFICANCE AND USE

This practice (F3079) is intended to assist engineers, contractors and owner/operators of underground utilities and tunnels with the successful implementation of distributed optical fiber sensing. F3079 includes DOFSS applications for monitoring ground movements prior to construction for site planning and during new utility and tunnel construction and operation, as well as the impact of such ground movements on existing utilities. Before the installation of distributed optical fiber sensing begins, the contractor shall secure written explicit authorization from the owner/operator of the new tunnel/utility and the existing utilities allowing an evaluation to be conducted for the feasibility of distributed optical fiber sensing for monitoring the impact of the ground movements on their assets and to have access to certain locations of the asset and the surrounding ground space.

It may also be necessary for the installer to have written explicit authorization from applicable jurisdictional agencies. Engineers, contractors, and owners/operators shall also be cognizant of how the use of distributed optical fiber might interfere with the use of certain equipment or tools near the installed optical fiber sensing cable in some special situations. For example, repair activities may have to temporarily remove, relocate, or avoid the optical fiber cable. Engineers, contractors, and owners/operators should be cognizant of how installation techniques and optical fiber (OF) cable location and protection can affect the performance of OFSS.

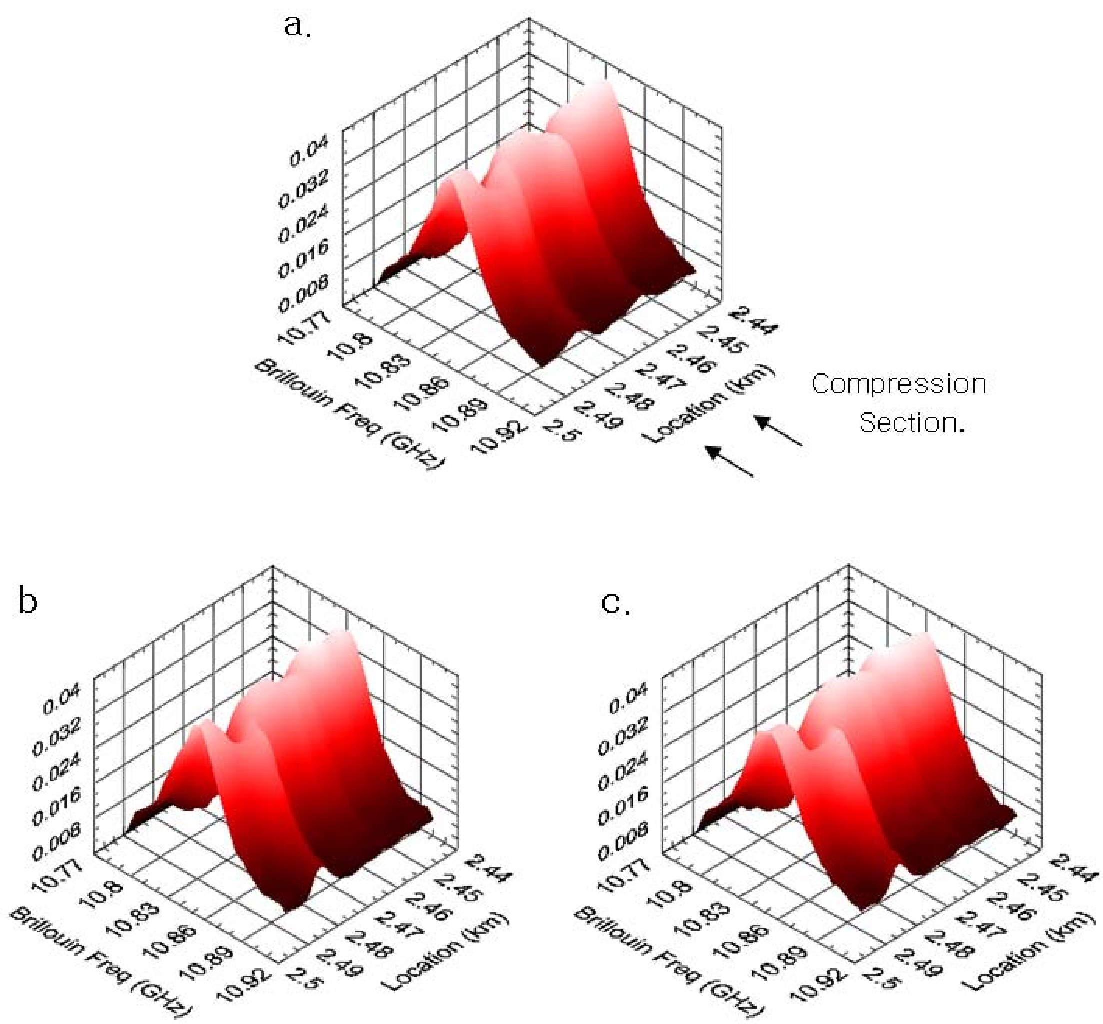

Figure 9. Typical results on strain measurements from BOTDA

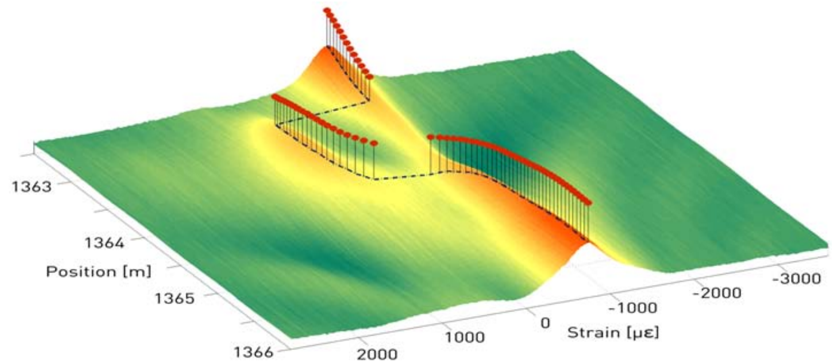

Figure 10. Typical results on strain measurements from BOFDA

MOST POWERFUL MEASUREMENT TECHNOLOGY

OFSS have many advantages over current methods using discrete "point" sensors for monitoring ground movements around underground utilities and tunnels. The advantages include, but are not limited to:

- their distributed nature means that there are no monitoring gaps, as compared to conventional point sensors, provided the distributed optical fiber sensing cable is installed over the whole length, area or volume of interest;
- a single optical fiber sensing cable can provide tens of thousands of continuously-distributed measurement points;
- no electricity used within the optical fiber sensing cable; thus, it is immune to electromagnetic interference and does not cause electromagnetic interference, other than that generated by the electro-optical equipment—which can be shielded and controlled;
- they are generally safe in explosive environments;
- they can be made robust to chemical exposure through proper design and materials selection for the protective outermost sheath of the cable;
- cost-effective due to the ability to collect data over long distances from a single electro-optical interrogator unit; cable lengths for a single system of 60 miles (100 km) are achievable.

Successful broader adoption of this technology depends on the proper selection of most appropriate materials, design, installation, data collection, interpretation and reporting user appropriate interface design. There are many different technologies that fall within the classification of DOFSS that can be used for measuring the impact of ground movement during tunneling or utility construction on existing utilities. The focus in this standard, however, is solely on the most widely used Brillouin scattering technologies (BOTDR / BOTDA and BOFDA). The DOFSS technologies discussed measure the longitudinal strains along the optical fiber sensing cables to enable the assessment of the impact of new tunneling and utility works on existing tunnels or utilities. The conversion of the strain measurements to displacement measurements requires processing of the strain data with appropriate assumptions for the boundary conditions as well as understanding the limitations. Therefore the resulting indirect displacement measurements are expected to yield an estimate of the in-situ displacements. As a result, the measured ground movements referred to in the text of this standard shall be used bearing this in mind. Methodologies for achieving better accuracy are also provided in the later section of the standard F3079.

PROBLEMS AND SOLUTIONS

Members of the Global OFSS Task Group live and work in Australia, Canada, China, France, Germany, Israel, Japan, Korea, Malaysia, Sweden, Switzerland, United Kingdom, and United States. Nevertheless, the distance or time zone did not matter; most of 40 plus members were participants of the second Wednesday of the month global teleconference held for no more than 30 minutes for everyone to share their knowledge and experience on solving problems that surfaced during the writing of these standards F3079 and F3092. Members had mutual respect for one another and serious disagreements on how to proceed with the content were worked through amicably by forming consensus among smaller subgroups holding their own teleconferences.

FUTURE PLANS

The Global OFSS Task Group has the following three standards in its future plans and with the following world renowned experts as chairs. Anyone with an interest to be part of these writing efforts, are most welcome to approach the chairs of these writing groups via email given here:

WK 43991: Standard practice for the installation of optical fiber cables along pipelines for leak detection using distributed vibration, strain, and thermal sensing: Chair of Core-Writing Team is Dr. Greg Duckworth, glduckworth@gmail.com

Proposed Scope: This standard will reduce pipeline and construction industry uncertainty on the selection and use of proper materials, locations, and methods for the installation of optical fiber cables along pipelines for the purpose of leak detection and localization. This standard will provide the information needed for best-practice on design and installation techniques to support leak detection using vibration, strain, and thermal sensing with optical fiber cables near pipelines.

WK 46887: Standard practice for the use of fiber optic distributed temperature sensing to detect leaks in above-ground ammonia, ethylene and LNG pipelines: Chair of Core-Writing Team is Dr. Daniele Inaudi, daniele.inaudi@smartec.ch

Proposed Scope: This standard will publish best practices on the selection and use of proper materials, locations, and methods for the installation, operation and maintenance of optical fiber cables for above ground pipelines transporting ammonia, ethylene, LNG and similar fluids for the purpose of leak detection using distributed temperature sensing (DTS).

WK 46971: Standard Practice for the Use of Optical Fiber Bragg Grating for Structural or Ground Monitoring: Chair of Core-Writing Team is Professor An-Bin Huang, huanganbin283@gmail.com

Proposed Scope: The use of optical Fiber Bragg Grating (FBG) as a strain sensor is gaining popularity. FBG can be attached directly to the surface of a target structure such as that of concrete or steel, as a strain sensor. Or, combining with other mechanical parts or chemical coatings, FBG can be used as part of a transducer for measuring a wide variety of physical quantities that include pressure, force, displacement, relative humidity or pH values, where the designated physical quantity is converted into strain and measured by FBG. Therefore the core to the success of this technology is for the FBG to function as a strain sensor that meets the required performance. Because FBG is partially distributive, multiple FBG or FBG based transducers can be connected via a single optical fiber. For ground monitoring where it is necessary to install the sensors underground, multiple and different types of FBG based sensors may be placed in a single borehole and result in much improved quality with reduced cost. Research and field experiments have demonstrated the feasibility and potential of this technology. One or more ASTM standards would be imperative for promoting the use of FBG globally. The new standard(s) shall include means and methods related to: (1) attachment of FBG to the target material either for direct strain measurement or as part of a transducer, (2) quality assurance for the attachment of FBG, (3) requirements for field set up of the FBG sensing system, and 4) collection and interpretation of data on various physical measurable quantities.

WK48360 New Practice for Standard practice for the use of optical fiber sensing systems for performing load tests and monitoring pile foundations supporting pipelines, conduits and utilities (Contact: Dr. Hisham Mohamad; mhisham@utm.my)

WK49252 New Practice for Spatial Resolution of Distributed Optical Fiber Sensors (Contact: Dr. Nils Noether; nils.noether@fibristerre.de)

WK49521 New Practice for Use of Optical Fiber Distributed Temperature Sensing Systems for Locating Illicit Connections on Sewers (Contact: Dr. Cedric Kechavarzi; ck209@cam.ac.uk)

BENEFITS OF GLOBAL STANDARDS

ASTM International offers an inclusive forum and the benefits from the development, publication and distribution of its standards around the globe help the optical fiber sensing industry and its users significantly. Given it being the oldest standard writing body in the world that has stood the test of time for 116 years brings instant recognition and credibility in front of those doubting the usefulness of OFSS. These standards reduce the amount of time it takes engineers to write bidding documents and technical specifications. The standards bring an added degree of comfort for the engineers, contractors and the users knowing that the thorough vetting process built as part of the consensus building within ASTM based on the balanced representation of consumers, users, producers and those of general interest. Standards help to form contracts between buyers and sellers.

In case of disagreements or disputes, standards form the backbone of establishing the "standard of care" in our judicial system. In a way, the buyers and sellers have the standards provide a preview of what case law is likely to be written and help them become aware of how to avoid errors and omissions. The biggest benefit of global standards is that these consensus documents pave the way to new technologies to become more widely accepted in the market place. Given the above benefits, writing standards is a worthy pursuit for all those who are willing to set aside their own personal interests and wish to give back to mankind more than what they have taken.

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Integrated Fiber Optic Sensing System for Pipeline Corrosion Monitoring

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Abstract

Corrosion significantly impacts the reliability and safety of metallic pipelines, which is a leading cause of metallic pipeline failure. A real-time update for the pipeline corrosion status and a timely alert for corrosion induced pipeline damages would contribute to an appropriate plan for pipeline maintenance and repair and reduce the frequency of pipeline failures. To assess the pipeline corrosion, various technologies exist and the most common approach is to measure the pipe-to-soil voltage potential. However, to date, few techniques can yet achieve remote and real-time corrosion assessment for pipelines. Fiber optic sensors, with unique advantages of real-time sensing, compactness, immunity to electromagnetic interference and moisture, capability of quasi-distributed sensing, and long life cycle, is a potential candidate to meet this challenge. This study, therefore, an integrated fiber optic sensing system is developed to assess the corrosion of on-shore buried metallic transmission pipelines in a real time manner. The sensing principle, development of embedment technique, and laboratory accelerated corrosion tests will be discussed in detail. Upon validation, the embedded integrated fiber optic sensing system could potentially serve the purpose of corrosion monitoring on numerous metallic pipelines and would possibly reduce the pipeline corrosion induced failures.

INTRODUCTION

Corrosion is the primary reason for metallic pipeline failure. According to U.S.DOT investigations, the average cost induced by corrosion management and related failure in U. S. was around $7 billion in 2002 (Koch 2002). For lowering the cost of pipeline maintenance, a timely corrosion assessment plays a significant role. Currently, the detection of pipeline corrosion has generally relied on qualitative visual inspections with the assistance of nondestructive evaluation tools.

There are several pipeline corrosion assessment technologies existing such as electrochemical, physical, and material sacrificial methods. The most widely used method is known as electrochemical approach. The electrochemical method measures the average corrosion effects by quantifying the electrical resistivity/potentials at the steel surface (ASTM 2008, Mansfeld 2003). Physical approaches also have been used for pipe corrosion detection as an indirect measurement method. It monitors the corrosion-induced structural degradations through measuring various physical quantities such as strain, guided wave, ultrasonic, and acoustic waves (Rathod 2006 & Steven 2007). Other than the two methods mentioned above for providing the average corrosion assessment, the material sacrificial approach is valuable to measure pin-point corrosion. It directly measures the corrosion-induced loss of materials by monitoring, for instance, the loss of coated metallic thin film materials (Qiao 2006, 2007, Leung 2008, and Wade 2008), the change in resistance/conductivity (Dickerson 2005) and the change in embedded metal antennas (Rathod 2006).

However, to date, limited approaches could perform real-time remote corrosion monitoring for pipelines in a cost-effective manner. Fiber optic sensors have unique advantages of multi-parameter and quasi-distributed sensing, long-term remote monitoring in real time, and low cost (Yu 2002). It is possible for the fiber optic sensors to be a cost-effective tool for real-time remote assessment of pipeline corrosion. Fiber optic sensors have been investigated for corrosion measurement based on material sacrificial and physical approaches. Material sacrificial based fiber optic sensors detect the light intensity changes from the thickness changes of the metal film coatings on the cleaved end of a fiber or out surface of an uncladded fiber. The metal films include Fe-C alloy, iron (Qiao 2006), electro less deposit of Ni-P, aluminum (Agarwala 2000, Abderrahmane 2001, & Benounis 2004), nickel (magnetic field vacuum deposition), and silver (chemical sputtering plating) (Abderrahmane 2001). The material sacrificial optical fiber sensor is simple but has a critical concern of multiplexing for monitoring corrosion in a large scale such as pipelines.

The physical based optical fiber sensors for corrosion measurement uses fiber grating techniques including Long Period Fiber Grating (LPFG) sensors and Fiber Bragg Grating (FBG). The LPFG based optical fiber sensors can assess the corrosion environments and relate that to the corrosion status such as moisture, pH, and metal ion sensors (Cooper 2001). Direct monitoring of the corrosion process based on the LPFG sensors also started recently (Huang 2013, 2014), but the protection of the sensors from damage during construction and service life become challenge for the LPFG sensors in practical application for pipelines.

The FBG sensors, for their superior stability and reliability, have been widely used for long-term strain sensing in civil engineering (Yu 2002, Zhang 2014, and zhou 2012). They can be applied for corrosion measurement through measuring corrosion induced strains. For example, FBG sensors packaged by Fiber Reinforced Polymers (FRP) (Zheng 2009) and Fe-C alloy (Dong 2007 & Hua 2009) were wrapped on the steel bar to measure the steel corrosion monitoring. However, the wrapping technique limited the corrosion assessment for a relative short period due to the maximum strain the FBG sensor can reach. Thus, there is a demand for new FBG sensor embedment techniques to monitor corrosion of pipelines.

In this paper, an integrated fiber optic sensing system is developed. Metallic coatings thermal sprayed on packaged FBG sensors is applied to embed the sensors on the pipeline for simultaneous corrosion assessment and mitigation. The embedment challenges for the FBG sensors to be inside metallic coating using thermal spraying coating process will be addressed and the laboratory accelerated corrosion tests will be performed to evaluate the developed technique.

FUNDAMENTALS OF CORROSION AND SENSOR DESIGN

Fundamentals of corrosion: The corrosion of iron is an electrochemical process which involves exchanges of electrons. With the presence of water and oxygen, the iron is oxidized to become ferrous ions which are prone to migrate to the catholic sites (Hua 2010):

$$2Fe^{2+} + 4OH^{-} \rightarrow 2Fe(OH)_2 \tag{1}$$

Further oxidation of Fe^{2+} to Fe^{3+} also exists with sufficient water and oxygen:

$$4Fe(OH)_2 + 2H_2O + O_2 \rightarrow 4Fe(OH)_3 \tag{2}$$

The hydrated ferric oxide is in an orange to red-brown color and it is the largest component of the rust products after corrosion process of iron based products. In addition, the ferric oxide also can be dehydrated into α-Fe_2O_3, which has a density much smaller than that of iron itself. Therefore, the rusting of iron product is normally accompanied by obvious expansion which can induce strains inside a metallic coatings to be measured by various strain measurement techniques.

Sensing principles of FBG sensors: FBGs are made by laterally exposing the core of a single-mode fiber to a periodic pattern of intense ultraviolet (UV) light, creating a fixed refractive index modulation, known as grating. At each periodic refraction change, a small amount of light is reflected, forming a coherent large reflection at a particular wavelength known as the Bragg wavelength. The Bragg wavelength satisfies the Bragg condition (Yu 2002 & Zhang 2014):

$$\lambda_{Bragg} = 2n\Lambda \tag{3}$$

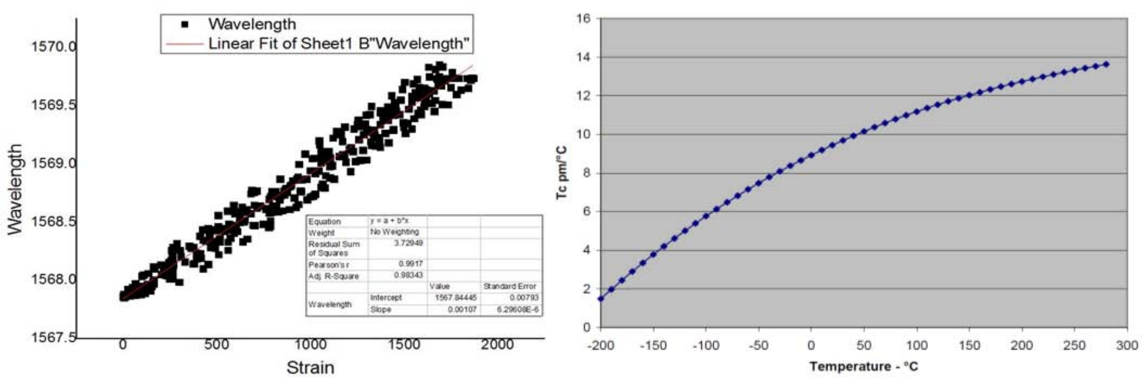
where, n is the index of refraction and Λ is the grating periodicity of the FBG.

Due to temperature and strain dependence of the parameter, Λ, the wavelength of the reflected component will change as a function of temperature and/or strain. The strain after temperature compensation can then be calculated as (Yu 2002 & Zhang 2014):

$$\varepsilon = \frac{1}{(1-P_e)}\left(\frac{\Delta\lambda_1}{\lambda_1} - \frac{\Delta\lambda_T}{\lambda_T}\right) \tag{4}$$

Equation (4) shows that if a FBG sensor is embedded inside a metallic pipe coating, when a corrosion process on the metal pipe coatings develops a strain change, the Bragg wavelength of the FBG sensors will change correspondingly to the corrosion induced strains. By tracking the strain gained by the embedded FBG sensors, the corrosion status can be monitored accordingly.

FBG sensors from Micro Optics, Inc. and the NI PXIe-4844 Optical Sensor Interrogator for data acquisition were used in this study. Sensor calibration tests were performed for both strain and temperature. Figures 1(a, b) show the strain and temperature calibration of the fiber optic sensor, respectively. The sensor has a strain sensitivity of 1.07 pm/με (pico-meter/micro-strain) and the temperature sensitivity of around 9.5 pm/°C (pico-meter/°C) in room temperature.



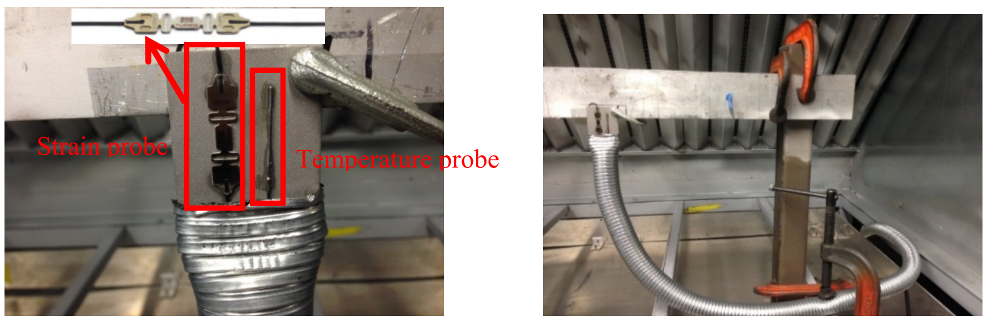
(a) Strain sensitivity (b) Temperature sensitivity

Figure 1. Strain and temperature calibration results for FBG sensors

DEVELOPMENT OF EMBEDMENT TECHNIQUE

Instead of attaching the sensors on the surface of the pipes, in this study, metallic hard coatings using thermal spraying coating process are used to embed the FBG sensors. Thermal Spraying (TS) Process is a general name for a group of deposition processes in which solid particles are melted and accelerated toward a substrate. The different methods of thermal spraying are classified based on the energy sources which are chemical and electrical (Bernecki 2004). Conventional Flame Spray and High Velocity Oxygen Fuel (HVOF) are among the techniques which rely on chemical energy as to melt the particles while Air Plasma Spraying (APS) and Wire Arc Spraying are examples of methods which are solely based on electrical energy. Thermal spraying coating is widely adopted for industries such as aerospace, automotive, bioengineering, marine, and civil structures (Arrabal 2010 & Aw 2008). The capability of applying a variety of coatings on to the different substrates has made thermal spraying an attractive industrial tool to protect, repair and manufacture of advanced structures and materials. Thermal sprayed metallic coatings had been approved to be an effective pipeline corrosion protection for off-shore pipelines (Antunes 2013).

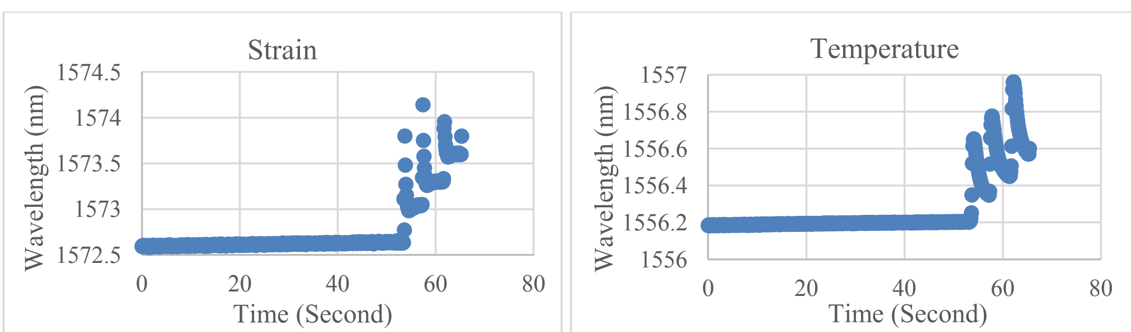
Direct Sensor Embedment in TS Metallic Coating: For a successful embedment, epoxy was used to temporary bond between the FBG sensors and subtract. Three FBG sensors were deployed including one bare FBG strain sensor, one steel packaged FBG strain sensor (Os3100), and one packaged temperature sensor. Figure 2(a) shows the sample. The communication fibers were protected using aluminum tubes to keep them damaged from the high-velocity wind as shown in Figure 2(b).



(a) Test sample layout (b) Coating scene

Figure 2. Test setup for the second trail

Figures 3(a, b) show the measured wavelength changes from the FBG sensors throughout the coating process. With a temperature sensitivity of around 9.5 pm/°C (pico-meter/°C), a maximum temperature can be obtained to be around 75 °C on top of the test sample throughout the coating process. The sensing system would survive the thermal spraying coating process since the maximum surviving temperature of the FBG sensors is more than 200 °C. However, the sensor failed after several passes of coating because of the direct contact of the high velocity stream from the coating. Fiber breakage was notified at the strain sensor as shown in Figure 4. Thus, it is required a more robust solution to avoid the expose of bare fibers to the high-velocity sprayed particles during the coating process.



(a) Obtained strain during coating (b) Obtained temperature during coating

Figure 3. Monitored wavelength changes of the strain and temperature sensors for second trail

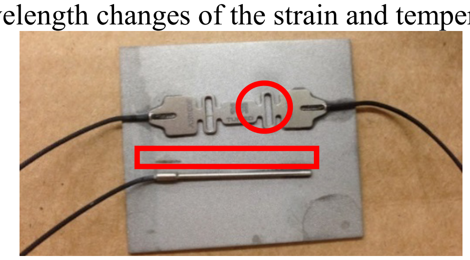
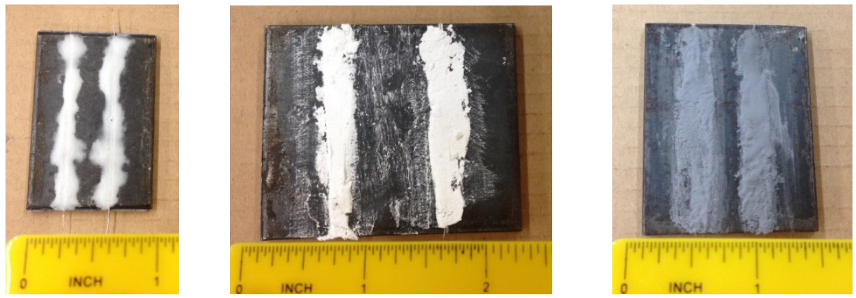
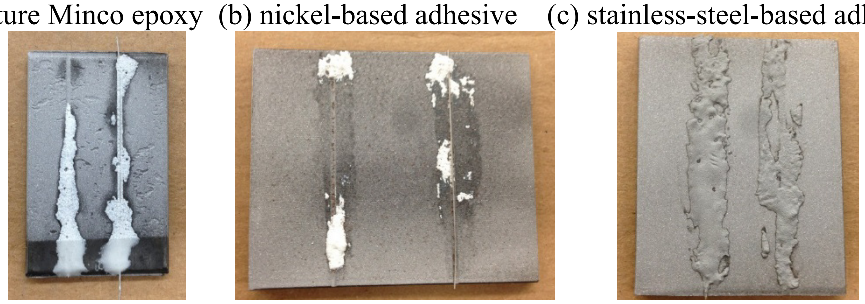


Figure 4. Sample after coating process

Protected FBG Sensors Embedded in TSMC: Protecting the bare FBG sensors from the high velocity of hard coating particles during the TS process, three different high-temperature adhesives (up to 1,000°F) were investigated for embedment, including 1) epoxy-based (Minco #15 Epoxy), 2) metallic-nickel-based (Durabond 952 from Cotronics Corp.), and 3) metallic-stainless steel-based (Durabond 954 from Cotronics Corp.). Figures 5(a~c) show the protected samples, respectively. Surface of all the tests samples were sandblasted before spraying to increase coating adhesion to the substrate as shown in Figures 5(e~g), respectively. After sand-blast, the protection of nickel-based adhesive failed. Thus, the nickel based metallic adhesive was eliminated for further consideration.



(a) High temperature Minco epoxy (b) nickel-based adhesive (c) stainless-steel-based adhesive



(e) High temperature Minco Epoxy (f) nickel-based adhesive (g) stainless-steel-based adhesive

Figure 5. Test samples before after sand blasting

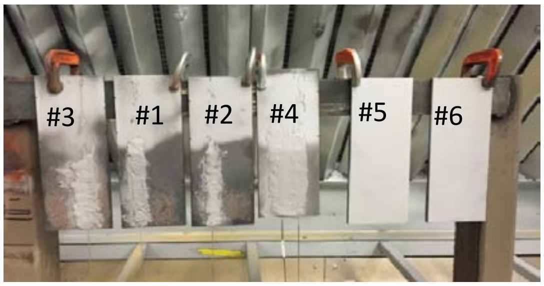
The two test samples (with epoxy and stainless-steel-based adhesive) succeeded the sandblasting were thermal sprayed using copper coating. Figures 6(a~c) show the samples after HVOF spraying process. The stainless-steel-based metallic adhesive successfully survived the thermal spray coating process as shown in Figure 6(b). The copper coating was successfully deposited on top of the adhesive and sensor at the thickness of 0.55 mm. With the successful embedment using stainless-steel-based metallic adhesive as protection on the sensor, all the FBG sensors will use this technique for embedment inside the TS metallic coating for further sensing capability study.

(a) High temperature Minco Epoxy (b) stainless-steel-based adhesive

Figure 6. Sample condition after coating deposition

SAMPLE PREPARATION FOR CORROSION TESTS

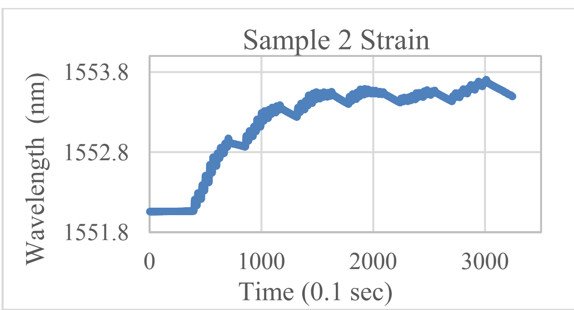
Base on the successful embedment technique developed, six samples were prepared for corrosion testing. The Cu-Al-Bronze was used as TS metallic coating for the samples. Figures 7(a, b) show the sand blasted samples before thermal spraying and during thermal spraying using automatic robotic spraying arms. A total of six traverses were made for the thermal spraying coating.



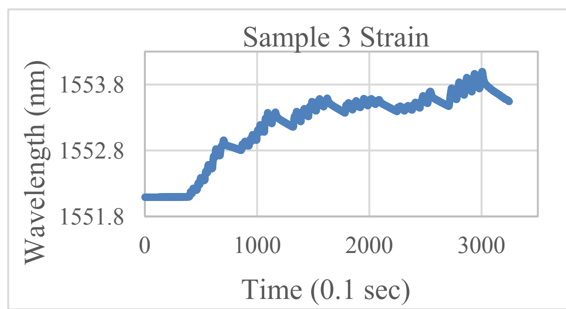
(a) Sandblasted sample setup (b) Thermal spraying process

Figure 7. Test samples and thermal spraying process

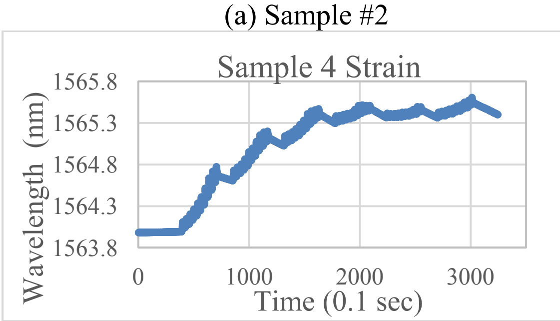
The sensor responses on all the samples were recorded during the thermal spraying process. Figures 8(a~c) show the center wavelength changes from the fiber optic strain sensors and Figure 8(d) shows that from the fiber optic temperature sensor. All the sensors successfully survived the thermal spraying coating process and monitored the coating process. The six traverses were clearly demonstrated in all the sensor readings. With the temperature increases during the coating, all the sensors performed consistently with temperature changes.



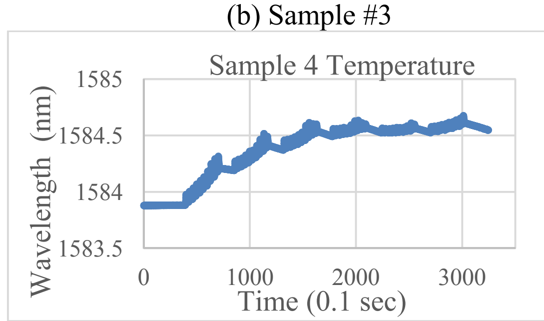
(a) Sample #2



(b) Sample #3



(c) Sample #4 strain sensor



(d) Sample #4 for temperature sensor

Figure 8. Sensor recording during the thermal spraying process

Table 1 listed each sensor's center wavelength changes right after coating and after cooling. An average center wavelength change of 1.425 nm was observed during the coating process for the fiber optic strain sensors and a center wavelength change of 0.663nm was observed for the temperature fiber optic sensor. With a strain sensitivity of 1.07 pm/με and a temperature sensor sensitivity of 9.5 pm/°C, Table 2 compares the resulted residential strain and temperature changes monitored during the coating process. It is indicated that an average of thermal strain of 1,329 με and a temperature increase of 70 °C was introduced by the elevated temperature during the coating process. After cooling down, the environmental temperature drops 6.3 °C before the coating and the coating had a thermal residual strain of an average of 192 με in compression with the entire processes considered.

Table 1. Sensor responses towards the HVOF thermal praying coating process

Sample No.	FBG Sensor Type	Center Wavelength before Coating (nm)	Center Wavelength right after Coating (nm)	Center Wavelength after coating Cooling (nm)
#2	OS1100 strain sensor	1552.064	1553.496	1551.812
#3	OS1100 strain sensor	1552.109	1553.544	1551.665
#4	OS1100 strain sensor	1564.002	1565.402	1563.874
#4	OS4210 Temperature sensor	1583.887	1584.55	1583.827

Table 2. Sensor response calculations

Sample No.	Wavelength Change right after Coating (nm)	Strain/Temperature Change (με; °C)	Wavelength Change after Coating Cooling (nm)	Strain or Temperature Change (με; °C)	Residual Strain Change after Temperature Compensation
#2 strain	1.432	1,338 με	-0.252	-235.5 με	-179.4 με
#3 strain	1.435	1,341 με	-0.444	-415.0 με	-331.0 με
#4 strain	1.4	1,308 με	-0.128	-119.6 με	-66.5 με
#4 temperature	0.663	69.8 °C	-0.06	- 6.3 °C	0 °C

ACCELERATED CORROSION TESTS AND RESULTS

With the samples prepared, accelerated corrosion tests using the developed integrated sensing system were performed by submerging PVC tubes on top of the samples with 3.5% NaCl solution for 21 days. Figure 9 shows the test setup with one example sample on top of the figure. The center wavelength changes of the four samples with embedded sensors had been recorded continuously for these 21 days with a sampling frequency of 10Hz.

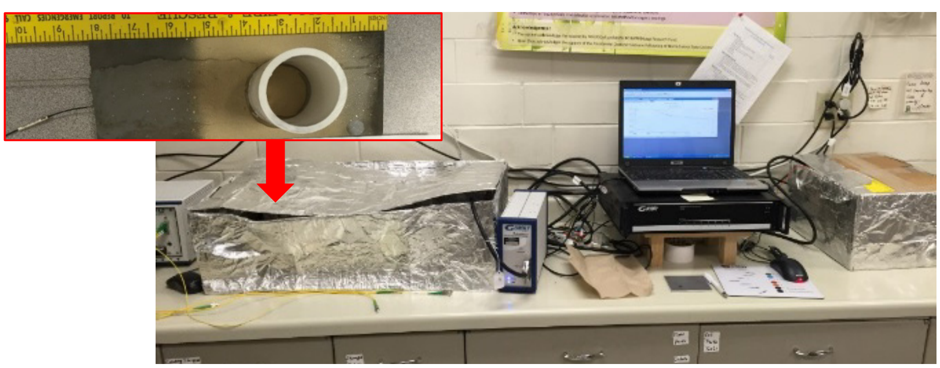


Figure 9. Corrosion test setup

Figures 10 (a~d) show the photos had been taken of each sample consistently during these 21 days for visual inspection. By comparing a sample each day from the photos, it is easily to figure out for Sample #1 and #2, the corrosion area was exactly above the embedded sensors. Since Sample #3 had been corroded before corrosion test, the corrosion area was larger than other samples. Sample #4 had less corrosion occurred which may be induced by a lower concentration of NaCl solution.

(a) Sample #1 (b) Sample #2 (c) Sample #3 (d) Sample #4

Figure 10. Corrosion visual inspection

Figure 11 shows the real-time monitored FBG center wavelength changes with submerging time after eliminating the temperature effect for the 21 days. All the four samples had an approximately same trend during the 21 days. Sample #3 was corroded before the 7-day test as shown in the bottom inset in Figure 11 for the sudden drops of the wavelength changes. So no further monitoring was performed on Sample #3 after 7 days. A total changes of 60pm for Sample #2 and 30pm for Sample #1 and 4 were noticed from the test results. After 15 days, consistently, all the samples were corroded into the coating as can be seen from the bottom right inset of Figure 11.

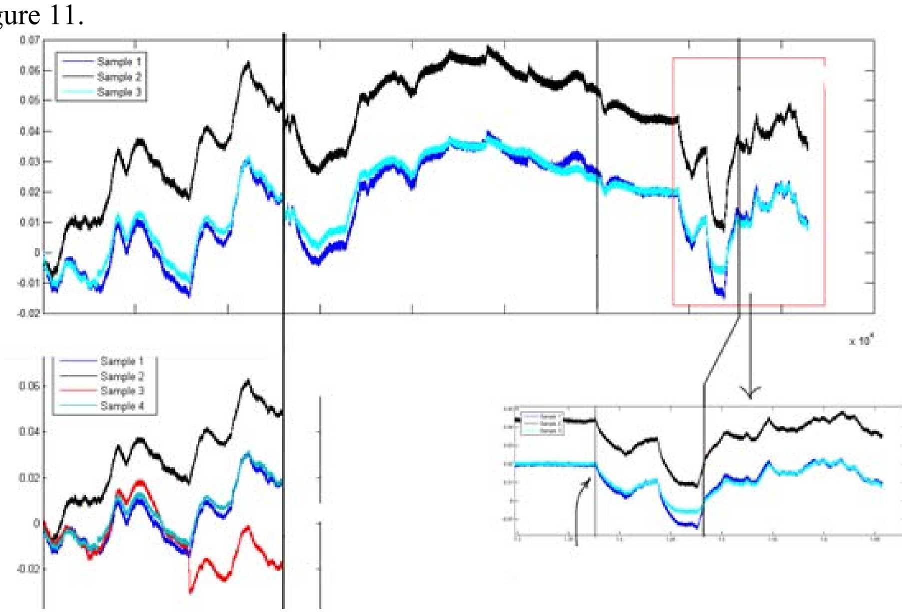


Figure 11. Center wavelength change from the sensing system from the accelerated corrosion test for 21 days

CONCLUSIONS AND FUTURE WORK

In this paper, an integrated corrosion sensing system was developed using thermal sprayed metallic coating to embed the FBG sensors. Challenges were noticed for a successful embedment of FBG sensors inside the metallic coating due to a high velocity and high temperature particle strings during the thermal spraying coating process. Through various trails tested in this study, a stainless-steel based adhesive protection layer on top of the sensor was validated to be an effective way of protection during sensor embedment. An average of thermal strain of 1,329 με and a temperature increase of 70 °C was introduced by the elevated temperature during the coating process. A residual heat strain of 193 με in compression was noted after the coating process. Six samples were prepared using the successful embedment technique and experimental corrosion evaluation testing was performed in laboratory. A qualitative corrosion measurement can be achieved using the developed integrated fiber optic sensing system to indicate the occurrence of the corrosion by extents. However, more analysis for a close correlation between corrosion induced metal loss to that of the sensor responses, which is on-going. Upon validation, the developed integrated sensing system could monitor the pipeline corrosion across nation and would possibly reduce the pipeline corrosion induced failures.

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Effects of Ultraviolet (UV) and Thermal Cycling on Polyurethane (PUR) Coated Water Transmission Pipelines

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Abstract

Exposure of aromatic polyurethane (PUR) elastomers to UV irradiation and moisture is known to cause changes in color and physical properties. This paper will discuss mechanisms by which these changes occur and the degree to which they affect bulk physical properties. Several studies have questioned the detrimental effects on the long-term performance of PUR coatings used on the exterior surfaces of water transmission pipelines. In this study, Atomic Force Microscopy (AFM), Scanning Electron Microscopy (SEM), Differential Scanning Calorimetry (DSC) and Dynamic Mechanical Analysis (DMA) was utilized to determine the depth and extent of the damage. Steel panels were coated with two experimental standard AWWA C222 compliant systems and weathered for 2000 hr in a Xe-arc Weather-o-meter. Validation of the analytical results was accomplished by analyzing weathered and non-weathered panels.

INTRODUCTION

The use of polyurethane coatings for the protection of steel pipes for use in transmitting potable water is a well-developed technique. These polyurethane coatings are designed to meet the industry standard of AWWA C222. As the request for a new water line is fulfilled, steel pipes are manufactured, coated, and then transported to the construction site for installment. The time frame from a pipe being coated to being placed in the ground can be months to over a year. During this time, the pipe is exposed to the environment, which includes ultraviolet light, moisture, and heat. Detrimental effect of ultraviolet light on aromatic polyurethanes is a well-documented event [1, 2]. The potential loss of coating integrity to protect the steel against corrosion before being buried has been a concern for the industry. The purpose of this study was to use the atomic force microscopy and other methods to help detect the level and extent of degradation within the coating from accelerated laboratory weathering.

Atomic force microscopy (AFM) is an excellent tool to characterize the effect of environmental weathering on polymer systems but has yet to become a common tool to characterize weathered PUR coatings. This non-destructive technique has the ability to look at mechanical properties at precise locations of polymer surface and near-surface regions with little preparation to the sample [3]. In terms of weathering, bulk characterization of the polyurethane is not always helpful for the prediction of the system's effectiveness in continued corrosion protection. It is difficult to determine the precise depth of damage done to the coating due to weathering. AFM PeakForce™ QNM™ analysis provides the resolution and contrast necessary to determine the depth of weathering in the coatings by monitoring elastic modulus throughout the coating. Surface and near surface regions can be tested with high spatial resolution.

For this study two experimental elastomer coatings, coating A and coating B, designed toward AWWA C222 approval, were used to study the effects of accelerated weathering on polyurethane coating for water transmission steel pipelines. The coatings differ in physical properties by the degree of flexibility in the coating. Increased rigidity was due to formulation design that increased cross-linking density. Samples were exposed to a high degree of accelerated weathering for 2000 hours while being monitored over the course of weathering.

Laboratory accelerated weathering is not directly comparable to real life weathering without decades of testing. However, it does allow for one to study weathering in a controlled environment. Previous studies have used accelerated weathering and real environmental exposure to study ultraviolet degradation in polyurethane coatings for steel pipelines. Some of these studies have shown that accelerated laboratory weathering of polyurethane coatings had the minor effect of lowering tensile pull-off failure stress 10% or less after 4 weeks of accelerated weathering [4]. To expand on these previous studies, experimental coatings weathered in a Xe-arc Weather-o-meter were analyzed because this method introduces UV exposure, thermal cycling, and water spray. AFM PeakForce™ QNM™ was used as a new technique to help study the degree of weathering in combination with the typical methods of thermal analysis and optical microscopy.

EXPERIMENTAL

Polyurethane components were prepared and sprayed onto 4.0 x 8.0 x 0.25 inch steel plates. Steel plates were steel grit blasted to a minimum surface preparation of SSPC 10 'near white' with a surface profile that exceeded the AWWA C222 requirement of 2.5 mils (63.5 µm). Surface profile was confirmed using a DeFelsko PosiTector SPG surface profile gauge and extra-course Testex Tape. All samples were sprayed within six hours of their surface being prepared. Coating thickness was measured using a DeFelsko PosiTector 6000 electronic coating thickness gauge. Coating A average thickness was 42 mil (1067µm) and the average thickness of coating B was 41 mil (1041 µm). The free film samples used in this study were prepared on the same day by spraying the coating onto clean mold-released polyethylene sheets. All samples were allowed to cure for a full seven days before any testing was preformed or accelerated laboratory weathering was started.

Gloss measurements were collected using a BYK-Gardner micro-Tri gloss meter at 60° and 85°. The instrument was calibrated each time to a reference standard of a black glass. Color measurements were measured in the L* a* b system using a X-Rite Portable Sphere Spectrophotometer model SP64. Gloss and color of every panel was measured in five spots and averaged for each panel at pre-determined times.

Accelerated Laboratory Weathering

A set of control panels was allocated for testing of coatings in unexposed state. Another set of panels were prepared and subjected to 2000 hours of laboratory accelerated weathering in a Xe-Arc Weather-o-meter. The weathering method was preformed to the ASTM 155 cycle 1 equipment using Q-Sun equipment. The cycle profile of this test was 102 minutes of UV exposure with black panel temperatures reaching 63°C. This was followed by 18 minutes of light and water spray in the chamber (air temperature not controlled during water spray). This cycle was repeated 1000 times over the course of weathering. The benefit to using the Q-sun Xenon Arc spectra is that it is representative of the complete sunlight spectrum which includes ultraviolet, visible light, and infrared energies. This method allows the panels to be tested for weathering damage from short UV as well as the longer wavelengths that result in color fading and color change. A daylight filter was utilized that gave a light intensity of 0.35 W/(m^2 - nm) at a wavelength of 340 nm. This daylight filter creates a spectra equivalent to direct noon summer sunlight and creates a worse case extreme weathering for the coatings being tested. Coated pipes sitting in the yard see the UV spectra at different angles as the sun traverses the sky. The time the coating sees direct noon sunlight is relatively low compared to the whole time a coated pipe sits exposed to the environment. In addition, panels saw water spray for 15% of the time during exposure. This could be considerable high compared to the moisture a pipe would experience sitting in a more arid environment like Texas.

During the course of the accelerated weathering, samples were removed and their gloss, color, and mass were recorded after every 500 hours of exposure. Samples were allowed to cool to room temperature before testing.

Post Weathering Sample Analysis

In the AFM analysis of this study PeakForce™ QNM™ was used as the imaging mode that allowed quantitative mapping of the material properties when used in conjunction with calibrated AFM probes. The AFM probe was vibrated at a low frequency (2 KHz) and scanned across the sample surface. A feedback loop monitored tip penetration into the sample and maintained a constant deformation. This minimizes tip / surface damage, and allows individual 'force curves' to be collected at each pixel of the image. Information collected from various regions along the tip deflection (that makes up the force curves) can be extracted to produce maps of a sample's Young's Modulus (MPa to GPa), Adhesion (pN to µN), Deformation (nm) and Dissipation (KeV) with resolutions exceeded only by TappingMode™ Phase images. Built-in ScanAsyst™ Technology allows the imaging force (setpoint) and feedback gains to be optimized to produce distortion-free images [5].

Optical microscopic examination of the coated steel coupon surfaces was preformed prior to AFM analysis. A small coupon from each of the supplied samples was cut free and embedded in acrylic resin for cross-sectional analysis. After grinding and polishing the coupons were examined using optical microscopy with differential interference contrast (DIC) objectives. EDS scans were performed on exposed and un-exposed surfaces.

Thermal analysis was performed using ~60 mil (1524 µm) free film polyurethane that was sprayed onto mold released polyethylene sheets. DSC analysis was performed using a TA instruments Q200 DSC with the ASTM E1356 heat/cool/heat method from -80ºC to 175°C at a heat rate of 10°C/min. TGA Analysis was performed using a TA Instruments Q5000 TGA. Samples were analyzed from 30ºC to 750°C in air at a heating rate of 10°C/min. DMA analysis was performed with a TA instruments RSA3 DMA using the dynamic temperature ramp method at a heating rate of 5°C/min.

RESULTS AND DISCUSSION

During weathering, the panels were monitored for gloss and color change. Gloss loss was observed after 500 hours of accelerated weathering. Coating A lost about 20-35% of its original gloss after 500 hours of weathering. After 2000 hours of weathering the coating's gloss diminished by 94-99.3%. Coating B experienced a gloss loss at 500 hours of 40-53%. After 2000 Hours, the gloss had diminished by an average 95%. There was an appearance that the gloss diminished faster for coating B than coating A, which could be an indication that formulation plays an effect in the degree of weathering. However, since gloss measurements are performed on the coating surface to only a couple mils below the surface these numbers can only be assumed to reflect the coating surface and not the bulk mass of the coating. Color measurements showed that yellowing was occurring in the surface of the coating. It should be noted that over the course of the weathering no significant difference was seen in the coating thickness. This indicates that the handling of the plates throughout the course of weathering period had minimal effect on coating being removed from the samples.

Table 1.1 and 1.2 – Gloss loss of samples at time points relative to gloss at time 0 hr.

Coating A								
Hours	**A**		**B**		**C**		**Free Film**	
	60°	**85°**	**60°**	**85°**	**60°**	**85°**	**60°**	**85°**
500	21.65%	19.85%	30.42%	26.84%	24.23%	23.78%	28.23%	34.63%
1000	84.71%	56.99%	88.84%	52.14%	88.38%	53.30%	85.90%	76.77%
1500	91.24%	66.20%	98.01%	84.08%	97.50%	87.63%	96.08%	84.80%
2000	94.40%	78.11%	99.33%	99.19%	99.39%	96.00%	98.11%	91.09%

Coating B								
Hours	**A**		**B**		**C**		**Free Film**	
	60°	**85°**	**60°**	**85°**	**60°**	**85°**	**60°**	**85°**
500	44.22%	46.76%	40.20%	53.14%	41.59%	36.41%	33.83%	43.26%
1000	97.08%	85.25%	94.61%	76.31%	98.37%	80.70%	95.75%	81.13%
1500	97.72%	87.41%	97.03%	92.90%	99.52%	95.08%	98.19%	88.68%
2000	97.06%	87.66%	98.16%	98.63%	99.26%	94.35%	98.92%	95.84%

Optical Microscopy

After the expiration of the accelerated weathering period, optical microscopic examination of coated steel coupon surfaces was performed. The PU coating of both control samples contained numerous bubbles visible at the surface. Some were open to the surface, and cross section analysis showed entrapped air present throughout the coatings. Examination of the exposed coating showed surface discoloration and degradation effects as expected. Micro cracks and/or splits were visible on the surface of coating B, but absent on coating A. General optical microscope examination suggests that all these cracks or splits originate at a bubble, presumably a near surface one. Images of exposed surfaces show many more visible bubbles probably due to erosion of the amorphous skin region at the surface. Representative surface images at 2 different magnifications (20x, 50x) are shown in Figures 1 and 2

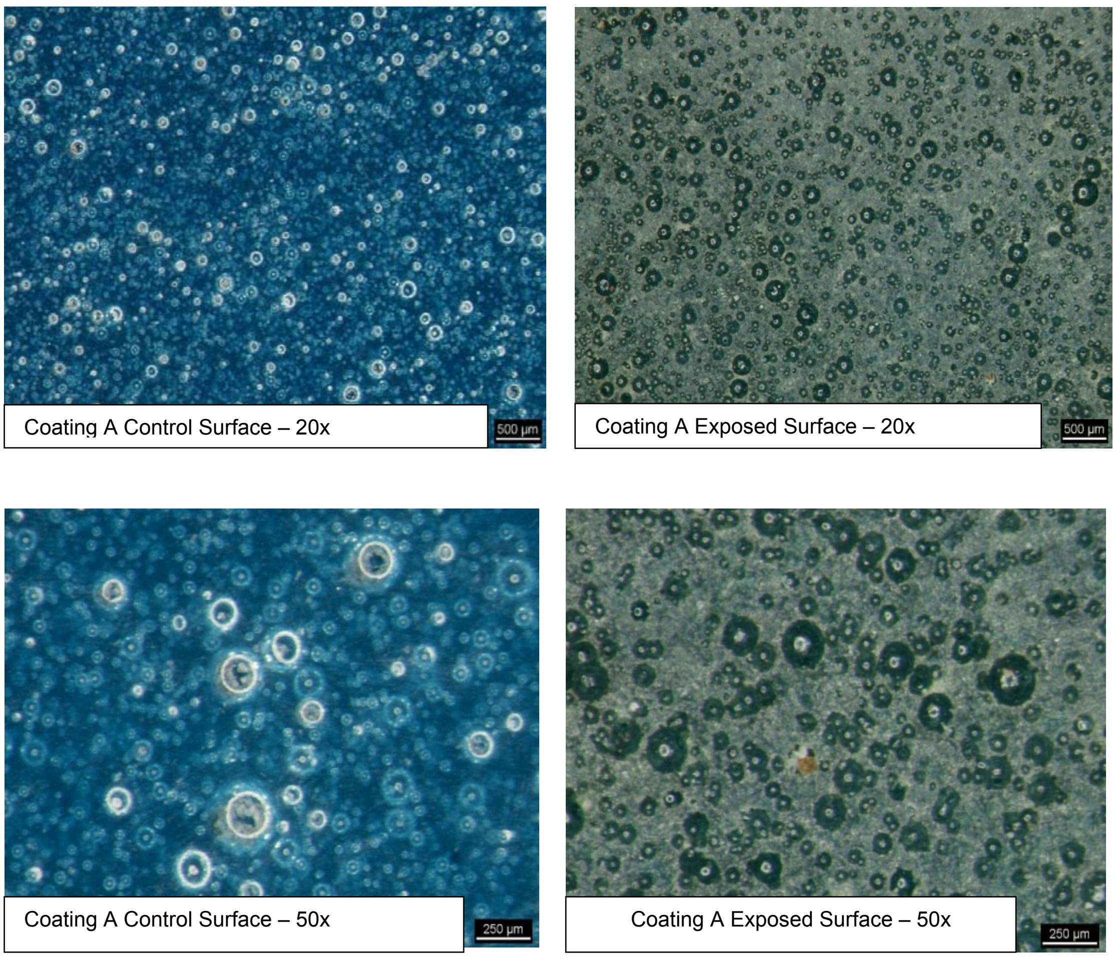

Figure 1. Surface images of Coating A Control (left) and Exposed (right) at 20x (top) and 50x (bottom) magnifications.

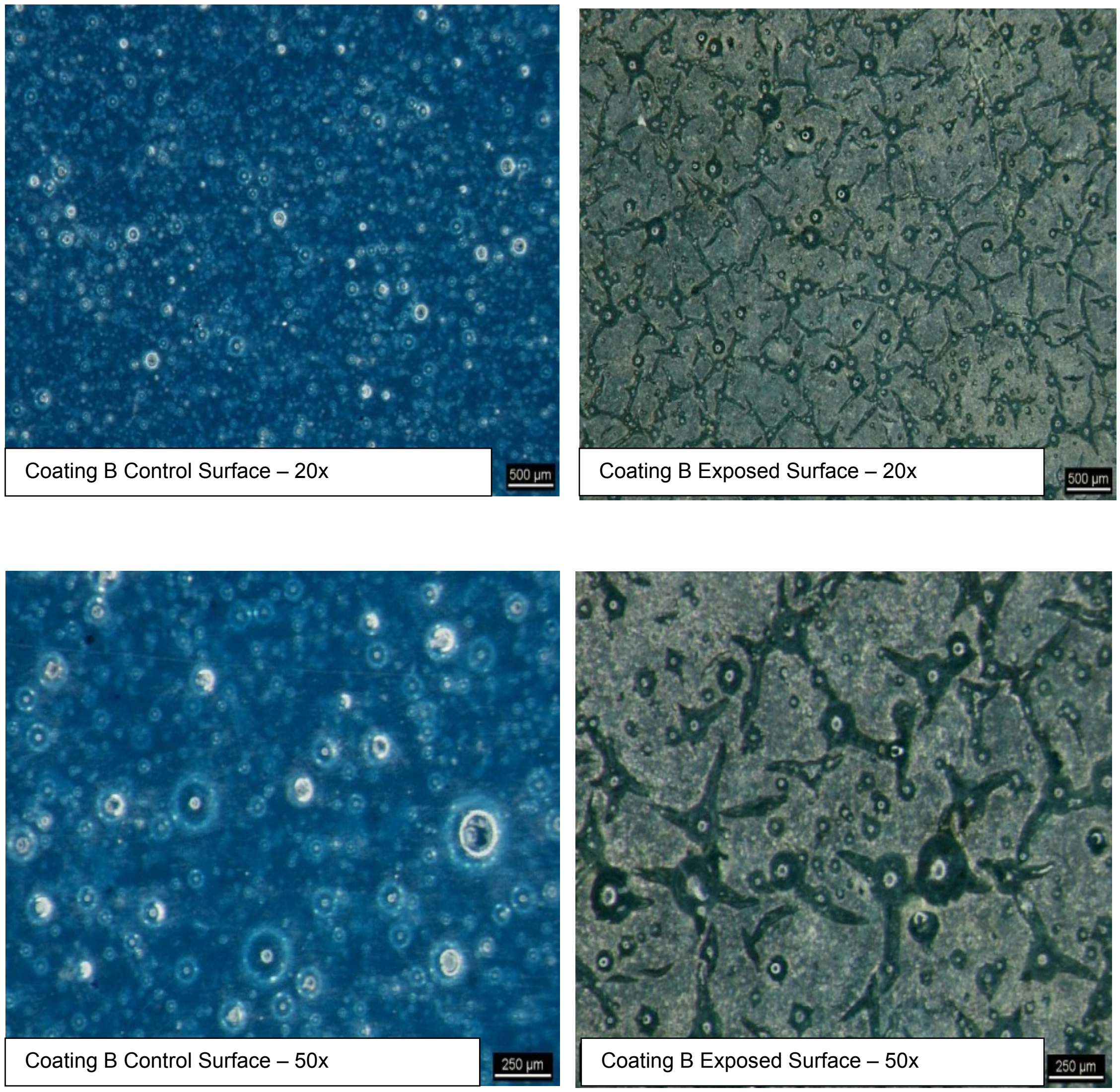

Figure 2. Surface images of Coating B Control (left) and Exposed (right) at 20x (top) and 50x (bottom) magnifications.

Cross sections of the exposed coatings showed a discoloration at the surface. However, this discoloration was observed only to a depth of approximately 2 mils (50 µm) in both coatings A and B. Energy dispersive X-ray spectroscopy (EDS) scans of the exposed coating surfaces from both panels showed a detectable increase in oxygen relative to the unexposed control surfaces.

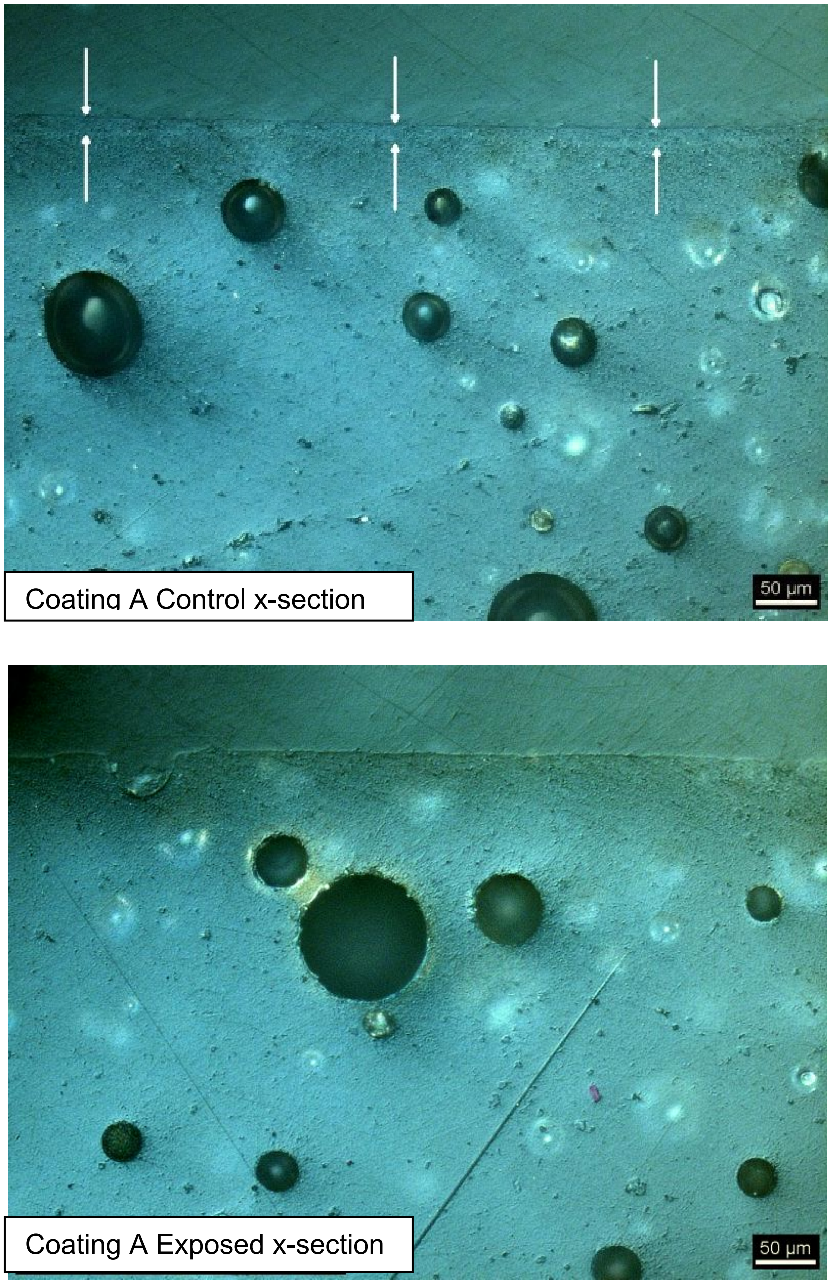

Figure 3. Images of Coating A Control (left) and Exposed (right) near surface cross-sections taken with polarized DIC illumination. Magnification = 200x. The coating thickness shown in images is the top ~20 mils (500 µm). Note the presence of a very thin 0.4 mils (<10 µm) skin layer at the surface of control sample (arrows). The exposed cross section shows the ~50 µm deep discoloration.

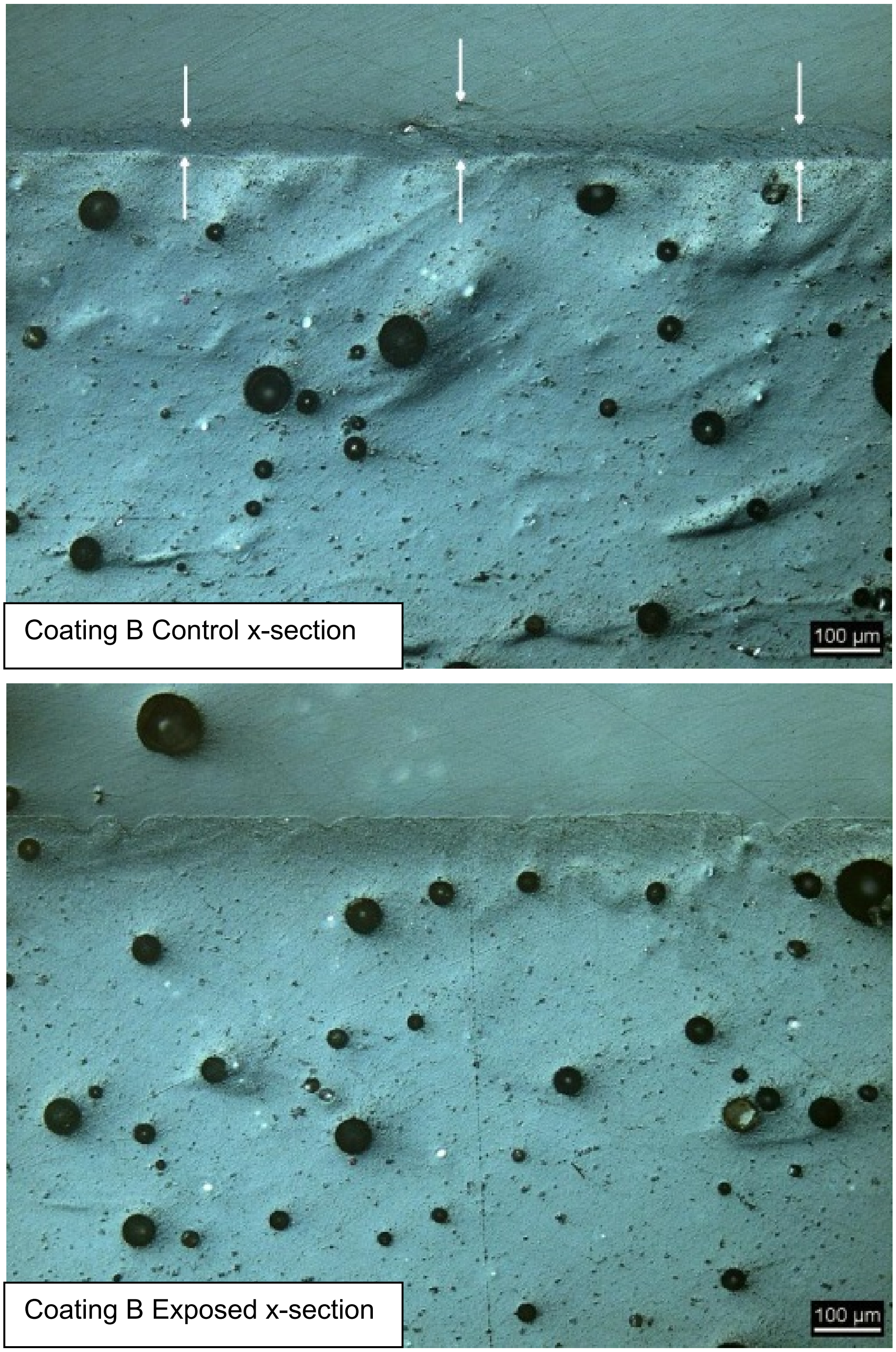

Figure 4. Images of Coating B Control (left) and Exposed (right) near surface cross-sections taken with polarized DIC illumination. Note the presence of a skin layer 2.0 mils (<50 µm) at the surface (arrows) and possible phase segments in the cross-section. Magnification = 100x. The exposed cross section shows the ~2.0 mil (50 µm) deep discoloration.

Coatings from each panel were then shaved from the metal substrate and mounted in AFM sample vises for Ultramicrotomy and AFM characterization. A glass knife was used to microtome 1µm thin sections from each panel so that a cross section view of the coatings could be performed. Figure 5 and 6 show reflected light microscope images of cross sections of A and B control (top) and exposed (bottom) coatings at the surface. When viewed using brightfield reflected light at 400X magnification very thin 'crusts' were observed at the surface of the exposed coatings. These crust layers measured 0.2-0.3 mils (5-7 µm) thick in coating A and ~1.0 mil (25 µm) in coating B. This suggests that depth of degradation in the coatings is limited to 5/1000 or 0.5% in coating A exposed and 25/1100 or 2% in coating B exposed.

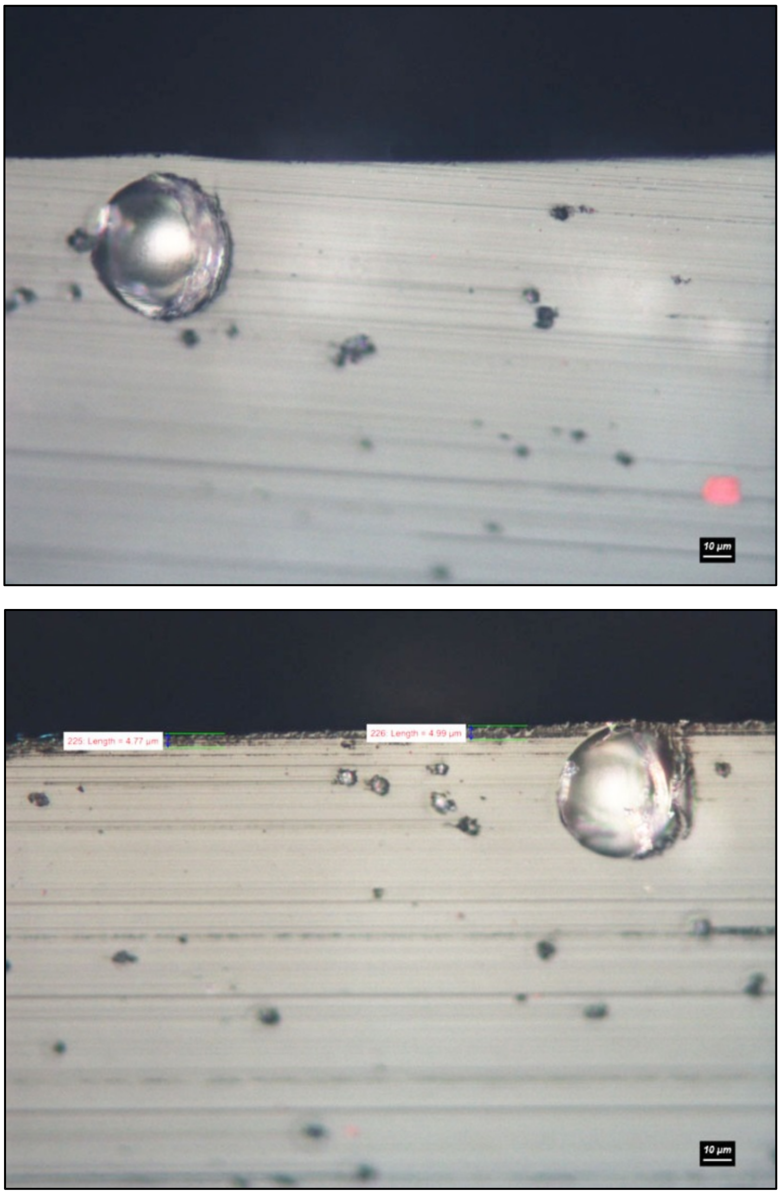

Figure 5. Brightfield reflected light microscope images. Coating A microtomed cross-sections of Control (top) and Exposed (bottom) coatings at the surface. Surface degradation shown in the Exposed cross-section is ~3.0 mils (4.8 µm) thick.

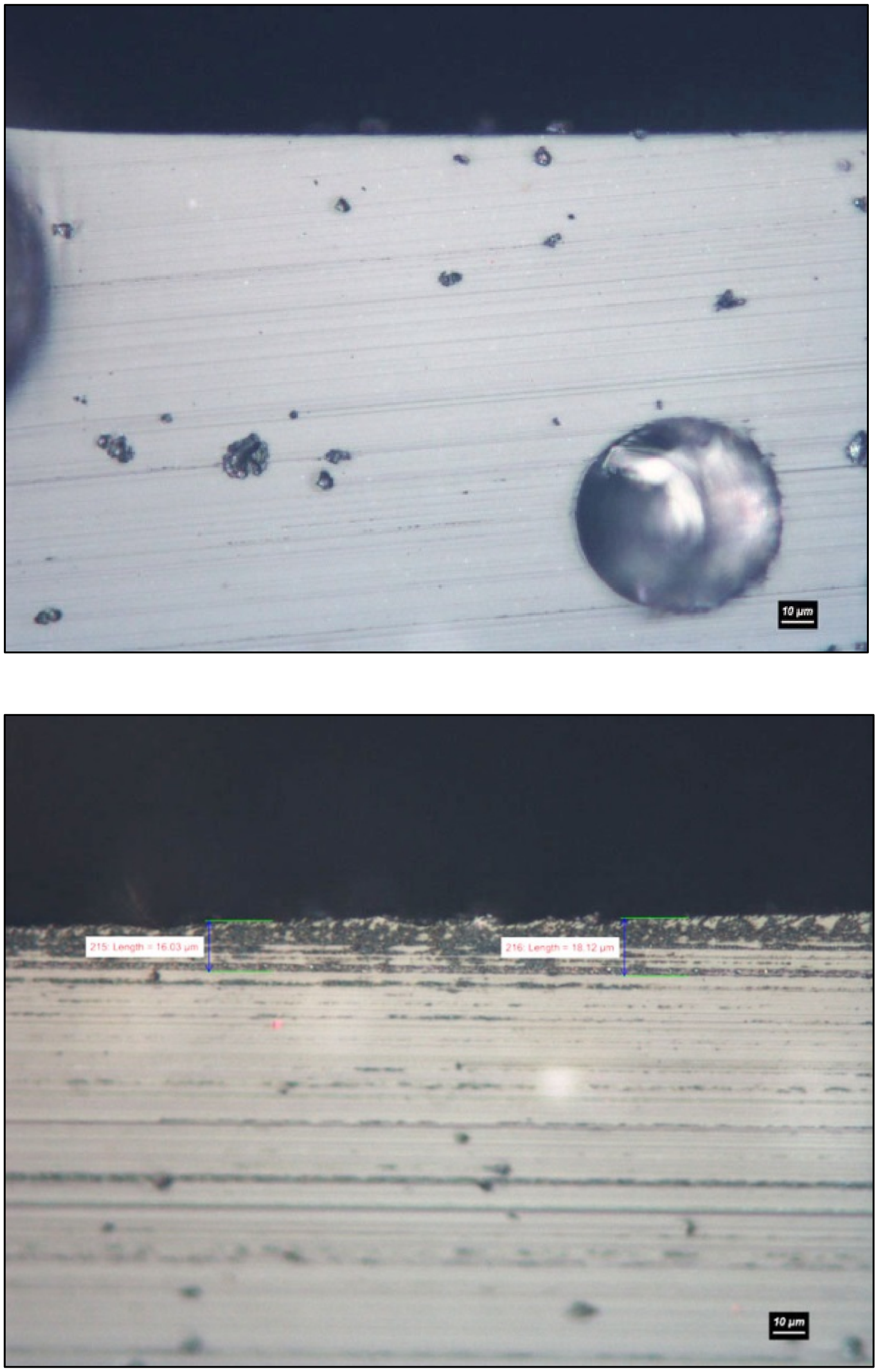

Figure 6. Brightfield reflected light microscope images. Coating B microtomed cross-sections of Control (top) and Exposed (bottom) coatings at the surface. Surface degradation shown in the Exposed cross-sections is ~ 0.7 mils (17 µm) thick.

Atomic Force Microscopy Analysis

Optical microscopic findings were verified using PeakForce™ QNM™ AFM which was performed on the microtomed cross sections of both the Control and Exposed coatings to compare Elastic Modulus (gigapascals) as a function of depth. In general, brighter regions in each map correspond to higher modulus, adhesion, deformation and dissipation. For this study a 10-color gray scale was used to map the Elastic Modulus values between 0 and 10 GPa.

Figure 7.1 and 7.2 show microtomed cross sections of coating A Control (left) and Exposed (right) coatings at 100X (left) and 400X (right) magnifications. Modulus maps were acquired at the surface (5µm x 5µm scans) and ~20 mils (500 µm) below the surface. For the Control coating the measured Modulus at the surface and below the surface was in the ~3.5 GPa range. For the exposed coating the Modulus at the surface measured ~7-8 GPa, but quickly relaxed to ~3.5 GPa within the first 4.0 mils (100 µm) below the surface (same as control). This indicates that the thin 'crust' layer at the top of the PU coating became more brittle with exposure to moisture and ultraviolet light. However, this change in the mechanical properties of the coating surface quickly disappears within 4.0 mils (100 µm) below the surface.

Figure 8.1 and 8.2 show similar Modulus maps for coating B Control and Exposed coatings. Of interest here is the 'apparent' softness of the Control coating at the surface (1-1.5 GPa) before increasing to ~3.5 GPa at ~4.0 mils (100µm) below the surface. This 'soft layer' was observed in the microscopy images shown in Figure 3 and is suspected to be the result of amorphous skinning. The exposed surface showed an Elastic Modulus of 9-10 GPa but quickly relaxed to 4-5 GPa at 2.0 mils (50µm) below the surface and back to the control sample modulus of~3.5 GPa at 4.0 mils (100µm).

Cross section measurements using PeakForce™ QNM™ AFM and optical microscopy indicate that depth of degradation in both exposed coatings is limited to the top 2-4 mils (50 -100 µm) of coating which equates to the top 5-10%.

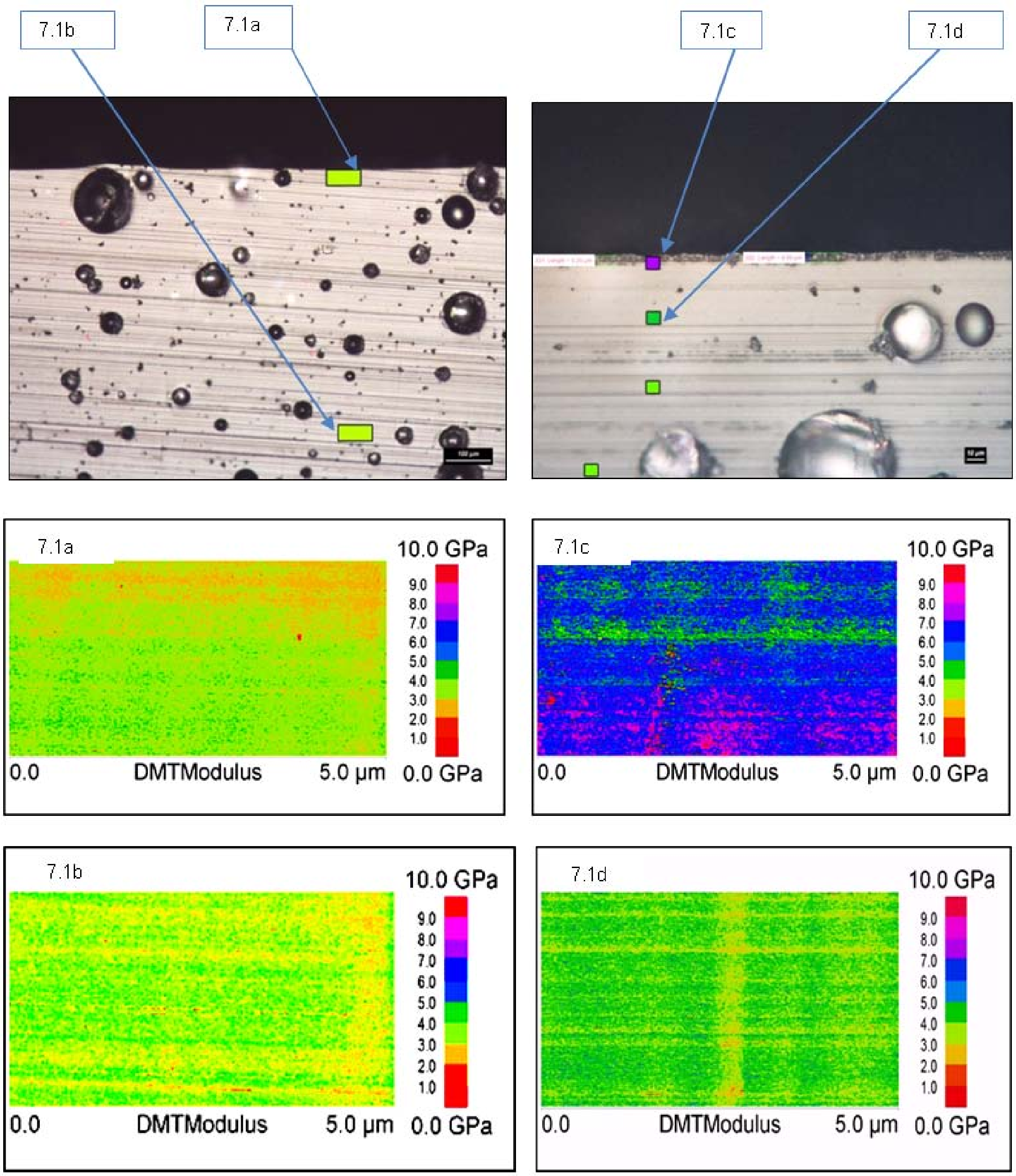

Figure 7.1a-d Coating A cross sections. Control (left) and Exposed (right). PeakForce™ QNM™ AFM Modulus maps taken at the surface and just below the surface to determine depth of degradation

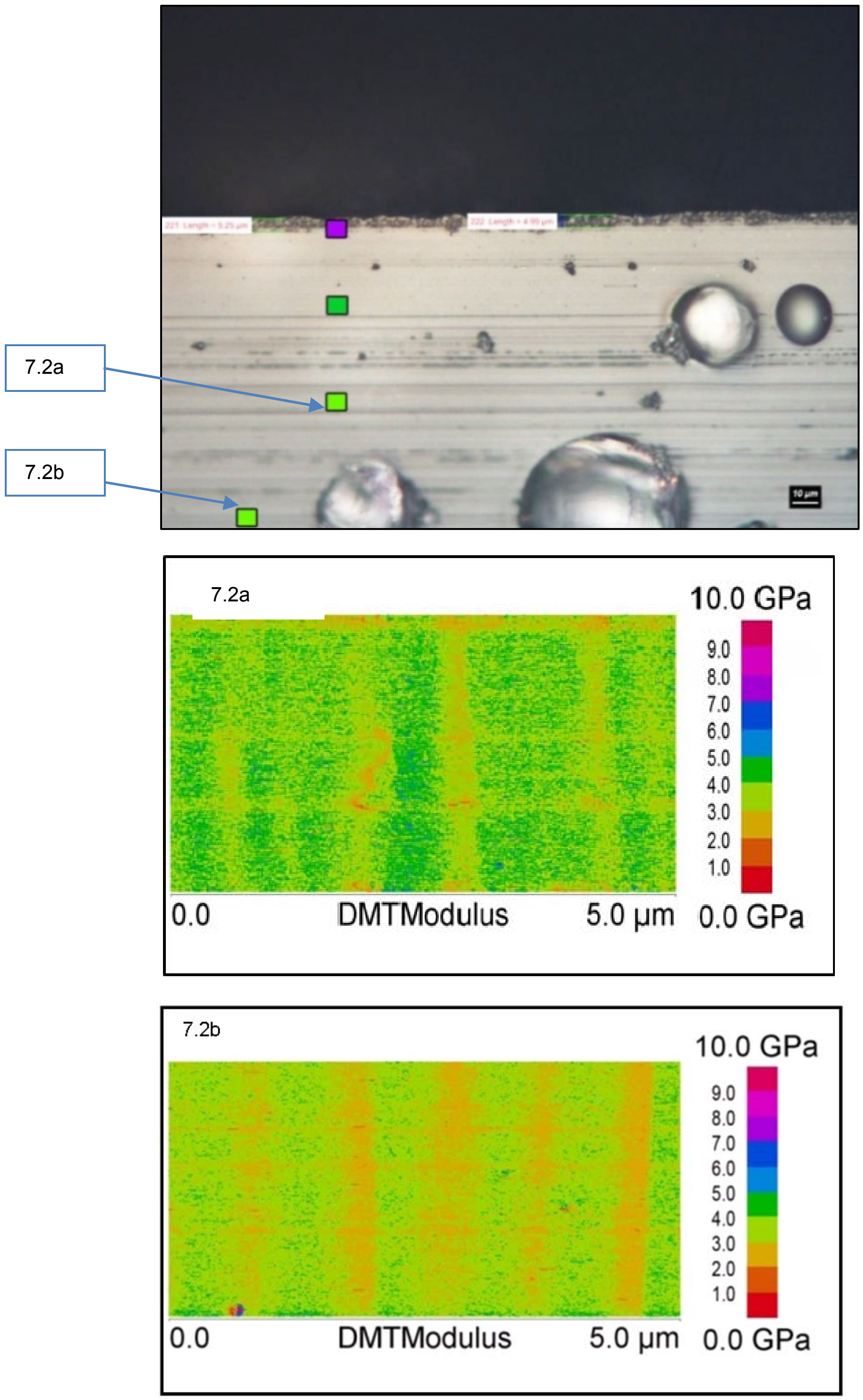

Figure 7.2a-b Exposed Coating A cross sections. PeakForce™ QNM™ AFM Modulus maps taken at below the surface to determine depth of degradation

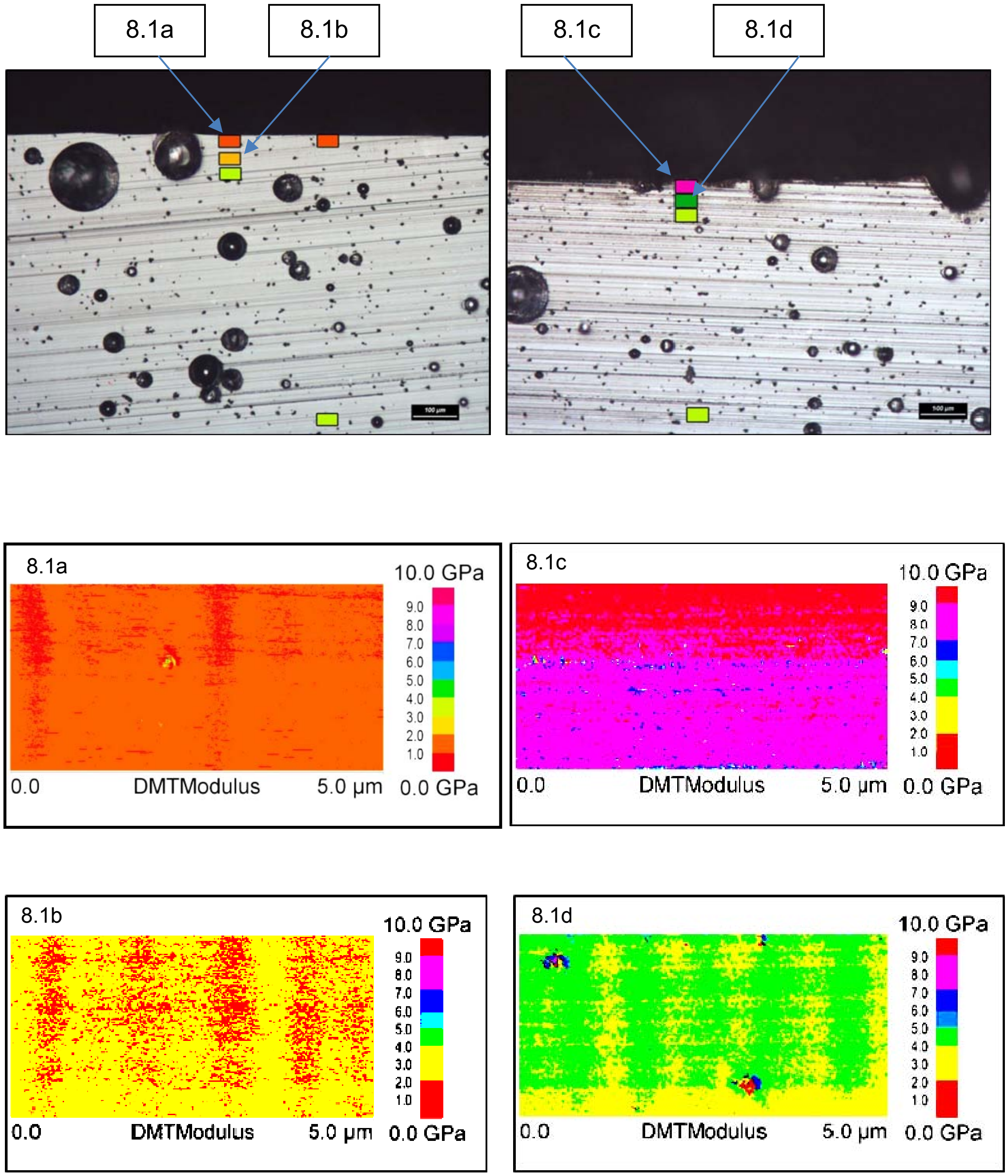

Figure 8.1a-d Coating B X-sections. Control (left) and Exposed (right). PeakForce™ QNM™ AFM Modulus maps taken at the surface the surface to determine depth of degradation. The control cross section shows approximately ~25 mils (670 µm) of coating thickness.

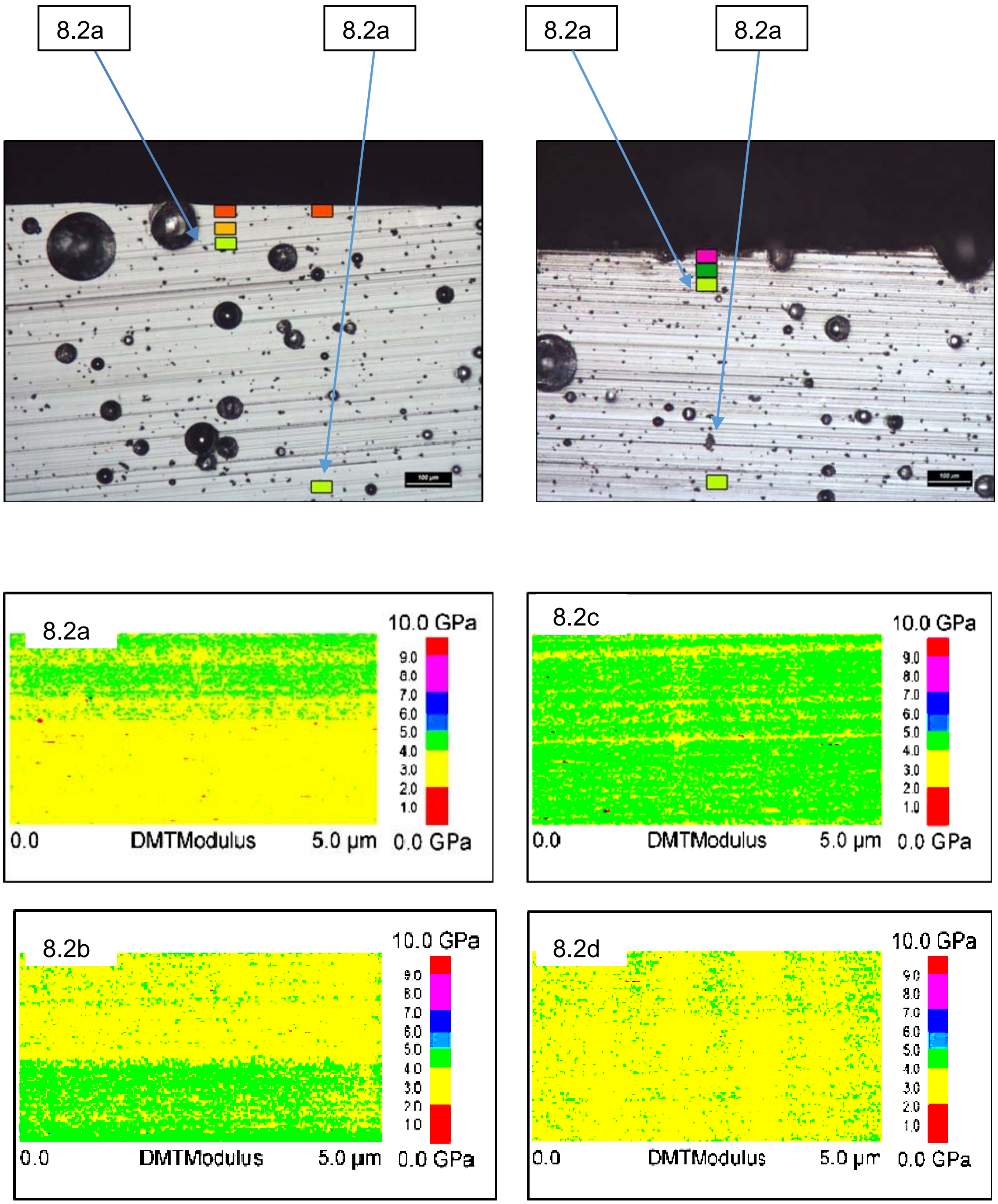

Figure 8.2a-d Coating B X-sections. Control (left) and Exposed (right). PeakForce™ QNM™ AFM Modulus maps taken just below the surface to determine depth of degradation. The control cross section shows approximately ~25 mils (670 µm) of coating thickness.

Thermal Analysis

Thermal analysis of the free films did not reveal significant information about any degradation of the coatings. TGA analysis was similar for all control and exposed samples with the thermal onset for degradation observed at 220-235°C. Dynamic mechanical analysis showed a minor difference in the control and exposed coatings with the Tg (°C) from tan delta increasing 14-15°C between the control and exposed samples. This difference was not seen in the Tg (°C) determined from the loss modulus curve. The increase in glass transition could be a result of post-cure occurring in the coatings with time. This increase in glass transition from differential thermal analysis was not detectable. Glass transition moments were difficult to resolve by DSC.

CONCLUSION

Atomic force microscopy has proven to be a useful analytical technique providing high spatial resolution to monitor depth of weathering in PUR coatings. Previous studies have utilized techniques that attempted to determine the depth of weathering but could not provide the resolution needed to determine coating degradation at the near surface.

Looking at all the factors and data generated from analysis of the weathered coatings, the degradation was limited to the top 5-10% of the 41-42 mil (1041-1066 µm) thick coatings. Though the degree of test exposure in this study could be classified as extreme, it is not considered a substitute for real time atmospheric exposure.

AWWA C222 specifies that coatings need to a minimum of 25 mils (635 µm). Most manufacturers spray a coating 7-10 mils (180-255 µm) thicker than the minimum specified. With maximum coating degradation being contained to the top 10% or 4 mils (100 µm) of the coating, one could justify as long as the coating thickness remains relatively thick at greater than 25 mils (635 µm), any degradation from exposure to UV light would be minimal and that the coating will continue to fulfill its purpose as a protective barrier. No matter what technique is used to analyze a coating to determine the effect of weathering the most important conclusion is if the coating can continue to protect the steel pipe after being exposed to the environment before being buried. This study has shown that even with extreme accelerated laboratory weathering chemical degradation of the polyurethane is contained to the top 10% of the coating. Further testing of weathered coatings is needed to determine if they are still able to meet all the requirements of the AWWA C222 specification. If a weathered coating can still pass all the specifications listed in AWWA C222 there is good indication that the weathered coating will still keep its integrity for pipeline protection.

ACRONYM LIST

AFM – Atomic Force Microscopy

DIC – Differential Interference Contrast

DMA – Dynamic Mechanical Analysis

DSC – Differential Scanning Calorimetry

EDS – Energy Dispersive X-ray Spectroscopy

PeakForce™ QNM™- Peakforce Quantitative Nanomechanical Property Mapping

PUR - Polyurethane

SEM – Scanning Electron Microscopy

TGA – Thermal Gravimetric Analysis

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Development of Constrained Soil Modulus Values for Buried Pipe Design

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Abstract

Simplified pipe design relies overwhelmingly on the empirical modulus of soil reaction, E', as a measure of soil stiffness which is back calculated from actual test results (Howard, 1977 and 2011). It is therefore not a measurable soil property. The relationship between the modulus of soil reaction, E' and true soil properties such as Young's modulus, Es, or the constrained soil modulus, Ms, has been investigated by a number of researchers. There seems to be a growing consensus that the modulus of soil reaction, E' can be directly replaced by the constrained soil modulus, Ms (McGrath 1998, and McGrath et al. 1999). Equipment for the triaxial compression test is not readily available to all soil testing laboratories especially temporary site laboratories, and the testing is relatively complex, expensive and time consuming. A relatively simple alternate to the triaxial test is the one-dimensional compression test or constrained modulus test. Although Ms is becoming the design soil stiffness of choice, very limited published data exists for buried pipe design whilst laboratory testing procedures are not well documented. It would be very beneficial to the pipeline industry to have a comparable set of soil stiffness values (i.e., Es and Ms) as a function of soil type, based on similar testing procedures and using the same soils. This paper presents the findings from an extensive geotechnical investigation conducted for a bulk water pipeline project involving over 390 triaxial tests and twenty constrained modulus tests. Following months of testing, data analysis and modifications, detailed guide specifications were developed for constrained modulus testing. Lastly, actual Ms values are presented for different soil groups and as a function of compaction density and stress level including recommended soil types for buried pipe backfilling.

INTRODUCTION

The design of large diameter pipelines is generally quite complex and requires careful consideration of the various stages of the pipeline including:

- manufacturing,
- transportation and delivery to site,
- off-loading and storage on site,
- installation and jointing in the trench,
- placing, spreading and compacting backfill material,
- temporary crossing of trenches with construction equipment,
- river and stream crossings,
- final reinstatement of trench and road layers, and
- fully operational conditions.

As a result, the structural design of the pipeline needs to evaluate the pipeline's structural capacity and ability to resist imposed loads and deformations during the different construction and operational stages. Once a pipeline is buried, it will be subjected to internal and external loads in addition to other potential loads.

Given the general variability of soil along a pipeline route and the fact that not all soil excavated from a trench may be suitable for pipe backfilling, proper material selection criteria should be established. Soil materials can be classified into soil stiffness categories (SC) as a function of soil type and intended use around buried pipelines as will be discussed in this paper. Such a classification should also provide specific information on the different soil types, recommended compaction densities, design stiffness, and general workability of the different soil types. This paper presents acceptable pipe backfill soil groups, compaction densities (as a function of design soil stiffness), use around pipelines, gradation requirements, and design soil stiffness values based on Ms values. It further presents suggested test procedures for determining constrained soil modulus values and compares actual test results to published data. The paper concludes with suggested design Ms values as a function of both compaction density and design soil cover (vertical soil stress), and recommendations for future testing.

PROBLEM STATEMENT

Equipment for the triaxial compression test is not readily available to all soil testing laboratories especially temporary site labs, and the testing is relatively complex, expensive and time consuming. A relatively simple alternate to the triaxial test is the one-dimensional compression test (1-D), constrained modulus test or oedometer test.

Although the constrained soil modulus, Ms is fast becoming the design soil stiffness of choice, few researchers have actually published design values for buried pipe design. Furthermore, actual laboratory testing procedures to determine Ms for typical pipe backfill soils are not well documented and are very limited. Lastly, it would be very beneficial to the pipeline industry to have a comparable set of soil stiffness values (i.e., Es and Ms) as a function of soil type and based on the same testing procedures and the same soils used for testing. In a recent study a test procedure was developed to quantify constrained soil modulus values for gravel size particles varying between 19 mm and 38 mm (Gemperline, 2010 and 2011).

1-D COMPRESSION TEST

The 1–D test apparatus and the non-linear stress-strain relationship obtained from the 1-D test are illustrated in Figure 1. Because of the lateral constraint on the soil specimen, failure will not occur. Therefore, this test is only suitable for measuring soil stiffness and cannot be used to determine strength. The slope of the confined stress-strain curve is constrained modulus, Ms as illustrated in Figure 1, and the reciprocal is the coefficient of volume change, m_v. It can be seen that the constrained modulus, Ms, increases with increasing stress. This parameter directly represents the vertical compression of unsaturated soil deposits (such as compacted fill) in situations where lateral movement is restricted.

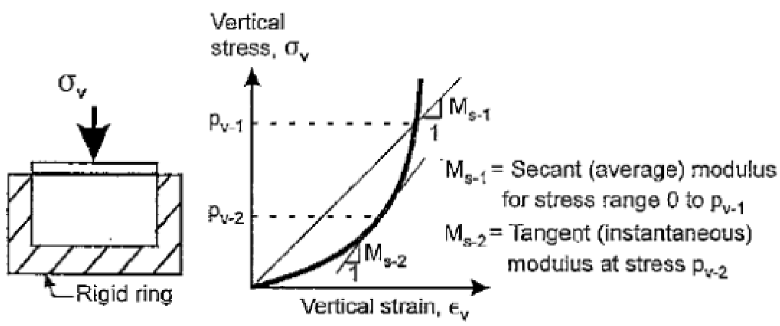

Figure 1. One-Dimensional Compression Test Configuration and Results

The one-dimensional compression test is not typically used for coarse-grained soils because the standard mold is small relative to the particle sizes, because of edge effects at the soil-mold interface, and because of difficulty in leveling the sample surface and getting uniform contact with the loading plates. Even though these problems are known to exist, the 1-D test is still considered a very useful and practical test to consider.

ACCEPTABLE PIPE BACKFILL SOIL GROUPS & STIFFNESS CATEGORIES

Table 1 summarizes typical soil material types as a function of pipe bedding type and use around buried pipelines. The soil classifications are grouped into Soil Stiffness categories (SC) based on the typical compacted soil stiffness values. Typical AASHTO (M145-91) soil types as well as USCS (ASTM D 2487-93) soil types are included in this table as well as any aadditional soil material requirements. Table 2 provides more specific information on the different soil types, recommended compaction density, design stiffness, and general workability of the different soil types.

Table 1. Typical Pipe Bedding Material Types.

Bedding Material Type	Use & Location [1]	AASHTO Soil (USCS Soil) [2]	Additional Requirements [3]	Soil Description
SC1A	Bed	-	P(13.2mm)=100% P(4.75mm)=80-100% P(2.00mm)=0-10% P(0.075mm)<2%	Crushed rock
SC1B	Bedding Cradle, Selected Fill Blanket	-	P(10mm)=100% P(4.75mm)=80-100% P(2.0mm)=0-20% P(0.075mm)<10%	Crushed rock
SC1	Bed, Bedding Cradle, Selected Fill Blanket	A-1-a (GW, GP)	P(10mm) ≤ 100%	Clean gravels
SC2	Bedding Cradle, Selected Fill Blanket	A-1-b (SW, SP, GM, SM) A-3 (SP)	- P(10mm) ≤ 100%	Clean, coarse grained soils
SC3	Selected Fill Blanket	A-2-4 (GM, SM) A-2-5 (GM, SM) A-2-6 (GC, SC) A-2-7 (GM, GC, SM, SC) A-4 (ML, OL)	P(0.15mm) ≤ 50% P(10mm) ≤ 100% PI ≤ 15 PI ≤ 15 P(0.075mm) ≤ 50%	Clean, coarse grained soils with fines, fine grained soils (silts)
SC4 [4]	In general, not acceptable as pipe Bedding Material	A-5 (OH, MH, ML, OL) A-6 (CL)	P(10mm) ≤ 100%	Silts and clays
SC5	Not acceptable as pipe Bedding Material	A-7-5 (OH, MH, ML, OL) A-7-6 (CH, CL)	- -	Clays
Soilcrete	Bedding Cradle	-	PI ≤ 10 P(10mm) ≤ 100% P(0.075mm) ≤ 25%	-

Notes:
1. Refer to descriptions below for clarification of pipe material zones.
2. Indicative USCS classification with respect to the AASHTO classification.
3. Maximum suggested particle size for all material types is 10 mm.
4. Bedding material type SC4 is in general not suitable for use as Bedding Material, but can be used as Selected Fill Blanket subject to further geotechnical evaluation and on the written approval of the Engineer.

Material specified in Table 1 above is defined as follows:

"**Bed**" means the zone in which bedding is placed and compacted over the full width of the trench, to a depth of 200mm minimum or as specified by the Engineer, on which a pipe or duct is placed such that the pipe is uniformly supported over the entire length of the pipe.

"**Bedding Cradle**" means the zone above the Bed in which bedding is placed firmly and without voids under and up both sides of a pipe or duct such that the pipe is uniformly supported over an arc length of 120°, up to the underside of the Selected Fill Blanket in a manner such that the pipe is uniformly supported.

"**Selected Fill Blanket**" means the zone above the Bedding Cradle in which material is placed and compacted to form a blanket around the pipe on or from the top of the Bedding Cradle up the sides and 300 mm over the top of a pipe, duct, or cable, in such a manner that the barrel of the pipe, duct, or cable is supported continuously and protected over the top by a dense cushion of Fill Blanket material.

"**Bedding Material**" means the material placed in the Bed, Bedding Cradle or Selected Fill Blanket.

"**Main Backfill**" means the approved filling material placed and compacted in the pipe trench after the pipe has been laid, bedded and surrounded by the completed Selected Fill Blanket.

Table 2. Summary of Suggested Soil Groups and Stiffness Categories for Pipeline Backfilling.

<table>
<tr><th>AASHTO Soil
(USCS Soil)</th><th>Extra Requirement for Backfill Soil</th><th>Min. Compaction,
% Std Proctor
(% Mod AASHTO)</th><th>Backfill Soil Stiffness Category (Description)</th><th>M_s, MPa (Below Water Table) [(1)]</th></tr>
<tr><td>A-1-a
(GW, GP)</td><td>-</td><td rowspan="2">90% SPD
(85% MOD)</td><td>SC1
(Crushed Rock; Clean Gravels)</td><td>23.7 MPa</td></tr>
<tr><td>A-1-b
(SW, SP, GM, SM)</td><td>-</td><td rowspan="2">SC2
(Clean, Coarse-Grained Soils)</td><td>18.2 MPa</td></tr>
<tr><td>A-3
(SP)</td><td>-</td><td>90% SPD
(85% MOD)</td><td>10.8 MPa</td></tr>
<tr><td>A-2-4
(GM, SM)</td><td>-</td><td rowspan="4">95% SPD
(90% MOD)</td><td rowspan="4">SC3
(Clean, Coarse-Grained Soils with Fines:)</td><td rowspan="4">16.6 MPa
(8.3 MPa)</td></tr>
<tr><td>A-2-5
(GM, SM)</td><td>-</td></tr>
<tr><td>A-2-6
(GC, SC)</td><td>PI ≤ 15</td></tr>
<tr><td>A-2-7
(GM, GC, SM, SC)</td><td>PI ≤ 15</td></tr>
<tr><td>A-4
(ML, OL)</td><td>$P_{200} \leq 50\%$</td><td>95% SPD
(90% MOD)</td><td>Fine Grained Soils: Silts</td><td>2.4 MPa
(1.2 MPa)</td></tr>
<tr><td>A-5
(OH, MH, ML, OL)</td><td>Not acceptable for backfilling</td><td rowspan="4">95% SPD
(85% MOD)</td><td rowspan="2">SC4
(Fine Grained Soils:
Silts & Clays)</td><td rowspan="2">2.4 MPa
(0.5 MPa)</td></tr>
<tr><td>A-6
(CL)</td><td>Not acceptable for backfilling</td></tr>
<tr><td>A-7-5
(OH, MH)</td><td>Not acceptable for backfilling</td><td>Not classified
(Fine Grained Soils: Clays)</td><td rowspan="2">2.4 MPa
(0.5 MPa)</td></tr>
<tr><td>A-7-6
(CH, CL)</td><td>Not acceptable for backfilling</td><td>Not classified
(Fine Grained Soils: Clays)</td></tr>
</table>

Notes: 1. Based on actual laboratory test results reported herein.

RECOMMENDED 1-D TESTING PROCEDURE

Following several months of laboratory testing, data analysis and modification of test procedures, detailed guide specifications were developed for constrained modulus testing using BS 1377-5: 1990 (BSi, 1990) as the basis with appropriate modifications to the following items [paragraph numbers in BS 1377-5: 1990]:

1) Loading sequence [Par. 3.5.1],
2) Consolidation time [Par. 3.5.2.4, 3.5.2.6],
3) Compaction density [Par. 3.3.1] and
4) Maximum particle and consolidation ring sizes [Par. 3.3.1].

Loading Sequence

1) Maximum Loading: 800 kPa.
2) Load Increments: 6, 12, 25, 50, 100, 200, 400 and 800 kPa.
3) Unload/Reload Cycle: Unload at 200 kPa following the same loading decrements to 25 kPa and continue loading as before for reload sequence.

Consolidation Time

Loading of the specimen can continue in steps with sufficient time between load increments to facilitate taking all measurements but not having to wait for any consolidation to take place as the specimens will for the majority of testing be conducted at optimum moisture content and therefore unsaturated. Also, the majority of soil samples selected for pipe backfilling will be coarse-grained / granular materials (35% or less passing 0.075 mm sieve) which will drain fairly quickly. Suggested (practical) time period between application of load steps is 10 minutes. Testing of fine-grained / silt-clay soils (more than 35% passing 0.075 mm sieve) and saturated specimens will take longer to consolidate and the recommended procedure of BS 1377-5: 1990 (BSi, 1990) should be adopted. The soil groups according to the AASHTO Classification system and suggested consolidation times are summarized below in Table 3.

Table 3. Suggested Consolidation Times as a Function of AASHTO Soil Groups.

AASHTO Main Group	AASHTO Soil Groups	Consolidation Time Unsaturated (Saturated) [(1)]
Coarse-Grained Soils ($\leq$35% passing 0.075 mm sieve)	A-1, A-2, A-3	10 minutes (10 minutes)
Fine-Grained Soils (>35% passing 0.075 mm sieve)	A-4, A-5, A-6, A-7	Par. 3.5.2.6 (Par. 3.5.2.6)

Note:

1) The above consolidation times are suggested time periods between successive load increments to allow for primary consolidation to take place and will vary as a function of the soil type. Highly compressible impermeable soils will take a lot longer to consolidate (i.e., 24 h to 48 h) while free-draining sand-gravel soils will consolidate almost immediately.

Compaction Density

Although BS 1377-5: 1990 (BSi, 1990) allows any compaction density to be specified, it is recommended that all specimens for the purpose of Ms testing be prepared at two compaction densities of 75 % and 85 % MOD AASHTO maximum dry density.

Previous testing at a very low density of 70% MOD AASHTO density resulted in large vertical strain (i.e., too compressible) and similarly, specimens prepared at 90% MOD AASHTO density are more difficult to prepare and result in smaller vertical strain (i.e., too stiff). However, it is believed that more representative results may be obtained by conducting the tests at two practical densities such as 75% and 85% MOD AASHTO and because the void ratio is known during testing, results can be determined for any compaction density in between and beyond 85% MOD AASHTO due to densification of the specimen during loading.

Maximum Particle and Consolidation Ring Sizes

The one-dimensional compression test is not typically used for coarse-grained soils because the oedometer ring is small relative to particle size, because of edge effects at the soil-mold interface, and because of the difficulty in leveling the sample surface and getting uniform contact with the loading plates. The recommended oedometer ring inside diameter is 50 mm to 105 mm while the ring height (H) should be at least 18 mm but not more than 0.4 x inside diameter (BS 1377-5: 1990). A typical oedometer ring size is 63.5 mm diameter x 25.4 mm height.

The recommended maximum particle size for the oedemeter test is H/5. Therefore, for a typical ring size of 63.5 mm x 25.4 mm the maximum recommended particle size is about 5 mm or say 4.75 mm (coinciding with standard sieve opening). In the pipeline industry a maximum particle size of 9.5 mm or 10 mm is typically specified to prevent coating damage resulting in a ring height of 47.5 mm to 50 mm.

According to BS 1377-5: 1990 for the maximum suggested ring diameter of 105 mm the maximum ring height is 0.4 x 105 mm equating to 42 mm or about 8.4 mm maximum particle height. A ring diameter slightly larger than 105 mm should work just as well especially for testing material with larger particle sizes as long as the suggested ring height to diameter ratio is maintained. Fortunately, the recommended largest ring size according to BS 1377-5: 1990 should suffice for testing most typical pipe backfill soils in practice. The code further allows for the removal of particles larger than the maximum particle size by passing the soil through the appropriate test sieve.

The suggested maximum ring diameter of 105 mm is believed to be based on practical considerations (i.e., capacity of loading apparatus and stack weights to create sufficient loading on specimen, undisturbed tube sample sizes, and testing of consolidation behaviour of fine-grained materials) more so than anything else and therefore, it is believed that for disturbed (recompacted) samples the maximum ring sizes can be increased.

The following ring sizes are typical examples of acceptable ring dimensions as a function of the maximum particle size, P_{max}:

- $P_{max} \leq 4.75$ mm : 63.5 mm x 25.4 mm
- $P_{max} \leq 8.5$ mm : 105 mm x 42.0 mm
- $P_{max} \leq 8.5$ mm : 120 mm x 40.0 mm
- $P_{max} \leq 10.0$ mm : 125 mm x 50.0 mm

1-D TEST RESULTS

Over forty constrained modulus tests were processed and analyzed for various soil types and blends some of which are summarized in Table 4 below. In general, most test specimens were prepared at 75 % and 85 % MOD AASHTO maximum dry density as discussed before.

Table 4. Summary of Processed Constrained Soil Modulus Tests.

AASHTO Soil Type	A-1-a		A-1-b		A-2-4		A-2-6		A-3		A-6	
MDD (kg/m^3)	2052		2058		2131		2113		1940		1953	
Target density of specimen, (%MOD AASHTO)	75%	85%	75%	85%	75%	85%	75%	85%	75%	85%	75%	85%
Initial dry density of specimen, (%MOD AASHTO)	67.1%	81.4%	73.9%	84.8%	73.7%	83.4%	73.7%	85.5%	74.9%	84.6%	74.5%	84.5%
Final dry density of specimen, (%MOD AASHTO)	80.5%	84.4%	78.9%	86.3%	79.2%	86.8%	86.0%	90.1%	83.4%	89.7%		100.9%
Initial void ratio, e_0	0.924	0.586	0.749	0.519	0.696	0.492	0.707	0.467	0.824	0.615		0.605
Final void ratio, e_0	0.603	0.531	0.639	0.459	0.521	0.434	0.485	0.392	0.639	0.523		0.345
Initial tangent modulus, Mi (kPa)	1395	2662	1776	4416	2044	3986	2453	4148	1490	2172	609	1436
Compression Index, C_c	0.267	0.069	0.112	0.068	0.162	0.068	0.192	0.067	0.125	0.086	0.380	0.230
Modified Compression Index, C_{ce}	0.139	0.043	0.065	0.045	0.096	0.046	0.113	0.046	0.069	0.054	0.186	0.143

Typical oedometer void ratio-vertical stress and axial strain-vertical stress test results are illustrated in Figure 2 for an AASTHO A-4 soil. The stress-strain behavior is non-linear and quite dependent on the maximum past pre-consolidation pressure. It is therefore important to distinguish between initial reloading, virgin compression loading, unloading and reloading.

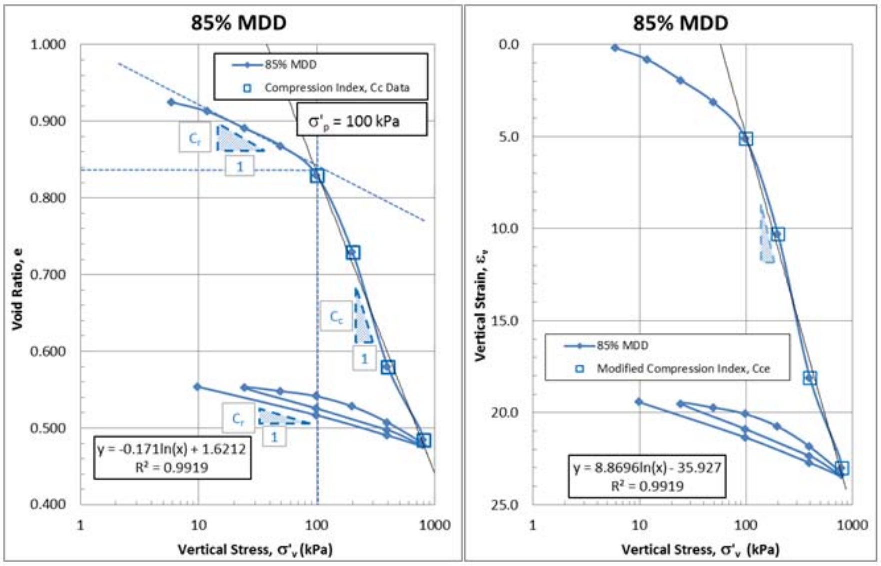

Figure 2. Typical Oedometer Test Results for an AASHTO A-4 Soil

Secant constrained soil modulus values were calculated for each test during virgin compression loading and plotted against both applied stress and specimen density (calculated from oedemeter test data as the specimen densifies during loading). Specimen density is easily calculated from vertical displacement, specimen diameter and mass of dry sample and can be expressed as a percentage of MDD. Ms-stress and Ms-%MDD values were plotted for each individual test (i.e., specimen prepared at 75% MDD) but also for each soil (i.e., combining 75% and 85% MDD test results). Excellent correlation was found between Ms and specimen density for all tested soils. The average correlation coefficient, r^2 for the twenty soils was as high as 84%. Figure 3 below illustrates typical results for an A-7-6 soil.

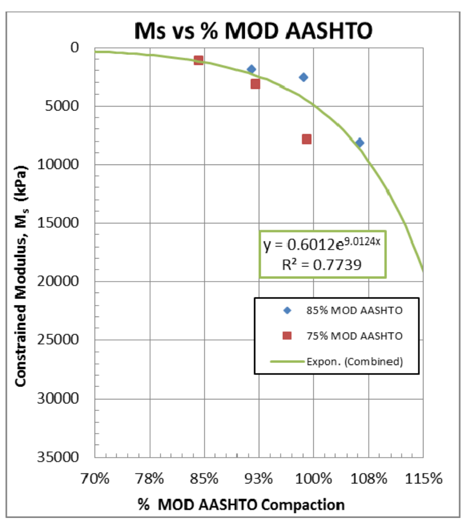

Figure 3. Typical Secant Constrained Soil Modulus vs Compaction Density

Figure 4 below presents secant Ms as a function of specimen density for the natural soils consisting of AASHTO A-1-a, A-1-b, A-2-4, A-2-6, A-3, A-6, A-4 and A-7-6 soils. It is very encouraging to note that secant Ms values increase with increasing soil strength and quality (i.e., A-1-a > A-1-b > A-2-4 > A-2-6 etc). This gives confidence in the testing procedures and data analysis. Similarly, Figure 5 presents the same results as Figure 4 but combined into a single graph for comparison purposes.

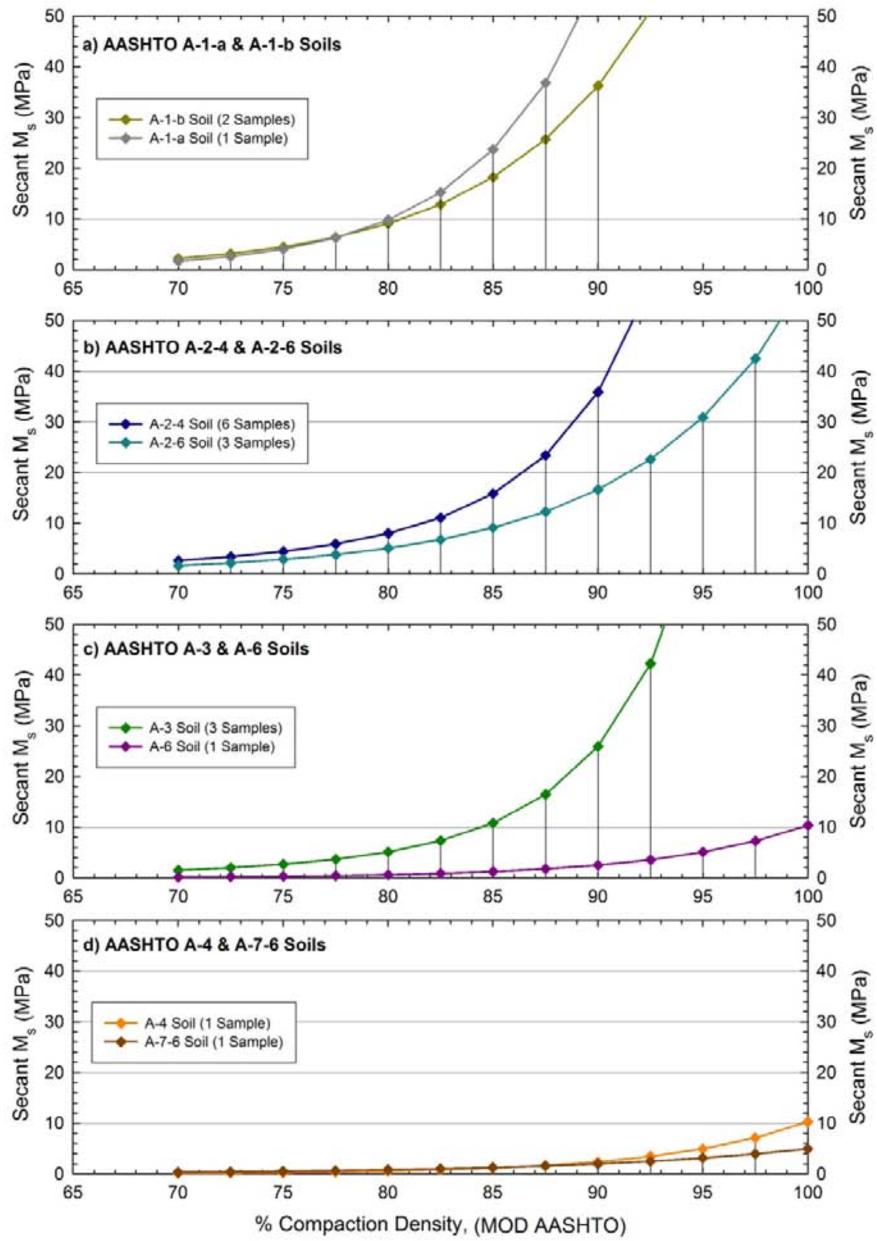

Figure 4. Constrained Soil Modulus Ms as a Function of Specimen Density

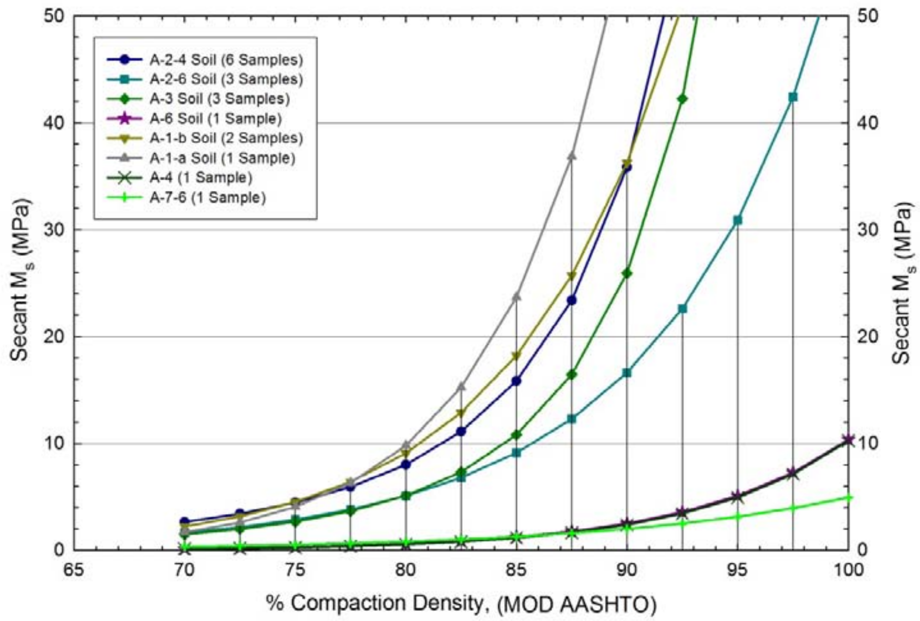

Figure 5. Secant Ms versus Specimen Density for All Soils Combined

From Figure 5, it is interesting to note that less acceptable pipe backfill soils such as A-3, A-6 and A-7-6 soils, show very little improvement in Ms with increasing density in agreement with Table 2. In fact, it is clear that A-7-6 soils should not be used for pipe backfilling as a maximum secant Ms of only 5 MPa is achieved at 100% MOD AASHTO MDD. Flexible pipelines will deform excessively when compacted with such poor material in an effort to reach 100% MDD.

The use of slightly better quality A-4 and A-6 material compared to A-7-6 material is also discouraged for several reasons including the generally low Ms values. A-6 soils (i.e., CL according to USCS) are inorganic clays of low to medium plasticity and are generally good to fair to work with in terms of ease of moisture-density control. A-4 soils on the other hand are generally inorganic silts and clayey silts and fair to very poor to work with based on ease of moisture-density control (i.e., very sensitive to moisture content).

All the other materials in Figure 5 will provide sufficient stiffness at practical densities of around 85 % MOD AASHTO and are considered acceptable for backfill material.

COMPARISON WITH EXISTING DATA

Figure 6 presents a comparison between calculated secant Ms values as a function of vertical soil stress (i.e., soil cover) and the University of Massachusetts recommended design values based on back-calculated results from triaxial compression testing (McGrath, 1998). Note that the latter data is not based on actual constrained modulus test results owing to limited test data and perceived testing difficulty.

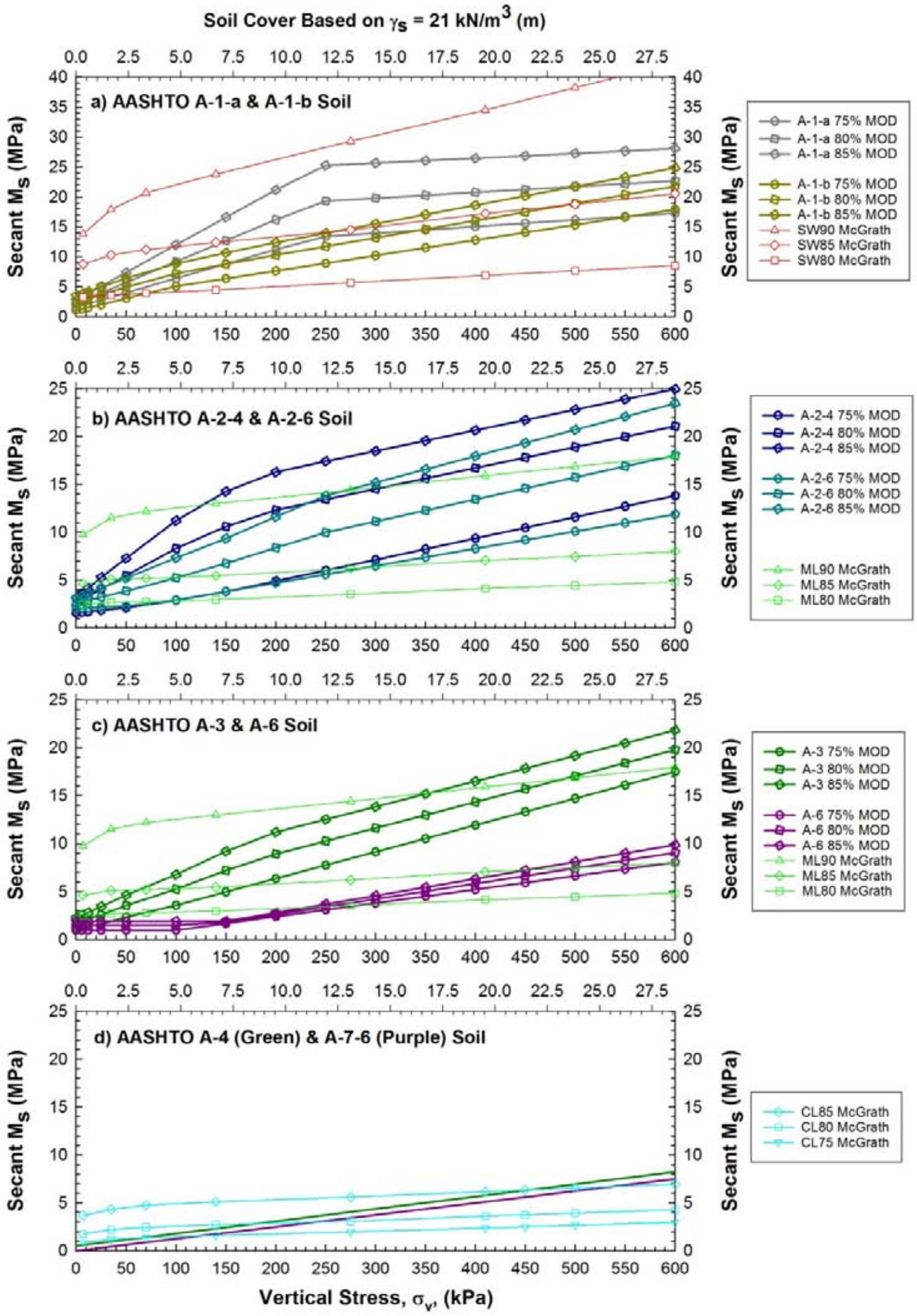

Figure 6. Comparison of Secant Constrained Modulus Ms with UMass Test Results

Figure 6 shows very encouraging and promising trends. First of all, the actual secant Ms values from this study fall within the the McGrath back-calculated results for the different soils groups (i.e., SW soil and A-1-a, A-1-b; ML soil and A-2-4, A-2-6, A-3 and A-6; and CL soil and A-4, A-7-6). Also, most of the coarser-grained soils show a bi-linear Ms-stress relationship in both data sets. A-4 and A-7-6 soils typically do not display this bi-linear relationship.

RECOMMENDED SECANT Ms VALUES

Table 5 presents a summary of proposed secant Ms values as a function of AASHTO soil type and compaction density. Note that these values are based on virgin compression loading in the oedometer apparatus up to 800 kPa.

Table 5. Proposed Secant Constrained Soil Modulus Values as a Function of AASHTO Soil Type and Compaction Density

%MOD AASHTO	Secant Constrained Modulus for Virgin Compression Loading (kPa)							
	A-1-a	A-1-b	A-2-4	A-2-6	A-3	A-4	A-6	A-7-6
70.0%	1680	2240	2660	1650	1500	130	150	340
75.0%	4060	4510	4470	2880	2680	270	310	520
80.0%	9810	9090	8020	5090	5120	560	610	820
85.0%	23700	18220	15850	9130	10810	1150	1240	1280
90.0%	57290	36270	35860	16640	25940	2390	2520	2010
95.0%	138500	71590	94920	30910	71050	4940	5100	3150

Similarly, Table 6 presents a summary of proposed secant Ms values as a function of AASHTO soil type and applied stress. Actual secant Ms test results were used to interpolate Ms values for 80% MOD AASHTO density and to adjust Ms values where the target oedometer specimen densities were slightly off target reported in Table 4.

Table 6. Proposed Secant Constrained Soil Modulus (kPa) Values as a Function of AASHTO Soil Type, MOD AASHTO Density and Stress Level

Proposed Secant Constrained Soil Modulus, Ms Values (kPa) for AASHTO A-1 and A-2 Soil Groups													
Soil Cover, m	Vert. Stress, kPa	A-1-a			A-1-b			A-2-4			A-2-6		
		75%	80%	85%	75%	80%	85%	75%	80%	85%	75%	80%	85%
0.0	0	1600	2200	2800	1090	2440	3510	1590	2570	3090	2030	2610	3120
0.3	6	1890	2620	3350	1310	2730	3860	1660	2930	3620	2060	2760	3370
0.6	12	2180	3040	3900	1530	3030	4210	1720	3300	4150	2090	2910	3620
1.2	25	2800	3950	5090	2000	3660	4970	1850	4090	5290	2160	3230	4170
2.4	50	4010	5700	7390	3060	4950	6430	2120	5480	7280	2300	3850	5220
4.8	100	6410	9200	11990	5090	7210	8880	2900	8310	11210	2910	5260	7310
7.1	150	8810	12700	16590	6370	8780	10670	3790	10600	14250	3810	6750	9340
9.5	200	11220	16200	21190	7660	10300	12390	4900	12310	16280	4710	8380	11620
11.9	250	13390	19320	25260	8940	11740	13950	6020	13430	17410	5600	9980	13820
14.3	300	13950	19800	25660	10220	13170	15500	7130	14520	18490	6500	11130	15200
16.7	350	14500	20280	26070	11500	14610	17060	8250	15610	19560	7400	12280	16570
19.0	400	15060	20760	26470	12780	16050	18620	9360	16700	20640	8290	13430	17940
21.4	450	15620	21250	26870	14070	17480	20170	10470	17790	21720	9190	14580	19320
23.8	500	16170	21730	27280	15350	18920	21730	11590	18880	22790	10080	15730	20690
26.2	550	16730	22210	27680	16630	20350	23290	12700	19970	23870	10980	16880	22060
28.6	600	17290	22690	28090	17910	21790	24850	13810	21060	24950	11880	18030	23440
42.9	900	20630	25570	30520	25600	30400	34190	20500	27600	31410	17250	24930	31680
47.6	1000	21740	26530	31330	28170	33280	37300	22730	29780	33570	19050	27230	34430
57.1	1200	23970	28460	32940	33300	39020	43530	27180	34140	37870	22630	31830	39930

Proposed Secant Constrained Soil Modulus, Ms Values (kPa) for AASHTO A-3 to A-7 Soil Groups											
Soil Cover, m	Vert. Stress, kPa	A-3			A-4			A-6			A-7-6
		75%	80%	85%	75%	80%	85%	75%	80%	85%	75% to 85%
0.0	0	1170	1740	2250	510	510	510	960	1480	1900	0
0.3	6	1270	1940	2530	580	580	580	960	1480	1900	70
0.6	12	1360	2130	2820	660	660	660	960	1480	1900	150
1.2	25	1600	2560	3430	830	830	830	960	1480	1900	310
2.4	50	2320	3530	4610	1150	1150	1150	960	1480	1900	620
4.8	100	3560	5250	6760	1790	1790	1790	960	1480	1900	1250

7.1	150	4960	7190	9190	2440	2440	2440	1670	1800	1900	1870
9.5	200	6350	8900	11190	3080	3080	3080	2380	2600	2790	2490
11.9	250	7750	10260	12520	3730	3730	3730	3090	3410	3670	3120
14.3	300	9140	11620	13850	4370	4370	4370	3800	4210	4560	3740
16.7	350	10530	12980	15180	5010	5010	5010	4510	5020	5440	4360
19.0	400	11930	14340	16510	5660	5660	5660	5210	5830	6330	4990
21.4	450	13320	15700	17830	6300	6300	6300	5920	6630	7210	5610
23.8	500	14720	17060	19160	6940	6940	6940	6630	7440	8100	6230
26.2	550	16110	18420	20490	7590	7590	7590	7340	8240	8990	6860
28.6	600	17500	19780	21820	8230	8230	8230	8050	9050	9870	7480
42.9	900	25870	27940	29790	12090	12090	12090	12300	13880	15190	11220
47.6	1000	28650	30660	32450	13380	13380	13380	13720	15490	16960	12470
57.1	1200	34230	36090	37760	15950	15950	15950	16550	18720	20500	14960

SUMMARY

The secant Ms values reported herein are one of the only sets of constrained modulus results that can be used for pipeline design based on actual oedometer test data. It is believed to be a major step forward in replacing the E' value (which is not a real soil property) with secant Ms values based on actual constrained modulus test results. Also, the test data was conducted to stress levels of 800 kPa representing soil covers of up to about 38 m. More testing is of course needed to expand the current soils data base. The UMass soils are quite broad (i.e., SW, ML and CL) whereas the current testing provides test results for several more soil types each well-defined using the popular AASHTO soil classification system.

Recommended pipe backfill material and use around buried pipelines were presented in tabular format whilst reported Ms test results show excellent agreement with the recommended backfill material relating to soil stiffness and compaction density. The effect of compaction density on secant constrained soil modulus values reduces with decreasing pipe backfill material quality. Conversely, for better quality coarse-grained soils (i.e., A-1-a to A-2-6 and A-3), the reported secant constrained soil modulus values are greatly affected by compaction density.

Coarse-grained soils display a bi-linear modulus-stress behavior with the breakpoint near the pre-consolidation pressures determined from oedometer testing.

In the longer term, it is envisaged that the amount of triaxial and hydrostatic compression tests (that are typically conducted on large-scale large-diameter bulk water pipeline projects) can be significantly reduced by conducting more Ms tests. In fact, it may be perfectly plausible to base new bulk water pipeline designs solely on soil classification testing coupled with the proposed secant Ms design values as opposed to conducting extensive triaxial compression and other soils testing (assuming a proper pipe design procedure is followed).

RECOMMENDATIONS FOR FUTURE TESTING

1) The following soil types should be included in future:
    a. A-2-5, A-2-7, A-5 group.
    b. Soilcrete should be tested because of its growing use and excellent pipe support properties (i.e., test at 4 and 8 hrs, 1, 7 and 28 days).

2) Additional constrained modulus testing should be conducted on similar soil groups to evaluate the following:
    a. Saturated soil conditions.
    b. Various practical soil blends with or without lime stabilization.

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The Value of Value Engineering—Functionality without Breaking the Bank on a Raw Water Transmission Project in Texas

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Abstract

The Tarrant Regional Water District (TRWD) with the City of Dallas Water Utilities (DWU), are currently engaged in the planning, design and implementation of a 350 MGD raw water transmission system, which will run across north central Texas from Lake Palestine to Lake Benbrook, with connections to Cedar Creek Reservoir, Richland Chambers Reservoir and a Dallas delivery point. Collectively, the system consists of approximately 145 miles of 84-inch to 108-inch pipeline, a 5-mile 120-inch diameter tunnel, three 150 to 275 MGD lake intake pump stations, three 200 to 350 MGD pump stations two of which include 80 MG suction reservoirs, one 450 MG balancing reservoir and ancillary facilities. The program developed by TRWD and DWU to accomplish these improvements is called the Integrated Pipeline Project (IPL). At the onset of the IPL, the project teams were given the mission to deliver this complex program in a sustainable manner by balancing the triple bottom line of "people, planet and profit" or "social, environmental and economic". As part of the efforts to optimize design functionality against the "profit" component a series of value engineering studies were conducted at key milestones along the project delivery schedule. Formal value engineering workshops were held at the end of the program's conceptual study to help guide further definition of project components and design scopes. The project was considered in three major areas: pipeline, booster pump stations and balancing reservoir facilities, and lake intake pump station facilities. Value engineering workshops for each of the components of design were conducted at preliminary design and progress design stages. Informal value engineering for each sub-project was collaboratively conducted through component concept reviews and milestone design reviews. The paper discusses the use and results of Value Engineering for a cross country water transmission line project with particular attention to the balancing reservoir sub-components of this system which through "VE" realized a range of savings of 30% to 50% of project budget on a delivered unit basis.

INTRODUCTION

The Tarrant Regional Water District (TRWD) with the City of Dallas Water Utilities (DWU), are currently engaged in the construction of the beginning phase of a 350 MGD raw water transmission system, which will run across north central Texas from Lake Palestine in east Texas to Lake Benbrook near the DFW metroplex, with connections to Cedar Creek Reservoir, Richland Chambers Reservoir, and a Dallas delivery point. Collectively, the system consists of approximately 145 miles of 84" to 108" diameter pipeline, a 5-mile 120-inch diameter tunnel, six 100–350 MGD pump stations two of which include 80-MG suction reservoirs, one 450 MG balancing reservoir, and ancillary facilities. The program developed by TRWD and DWU to accomplish these improvements is called the Integrated Pipeline Project (IPL). See Figure 1 for project location.

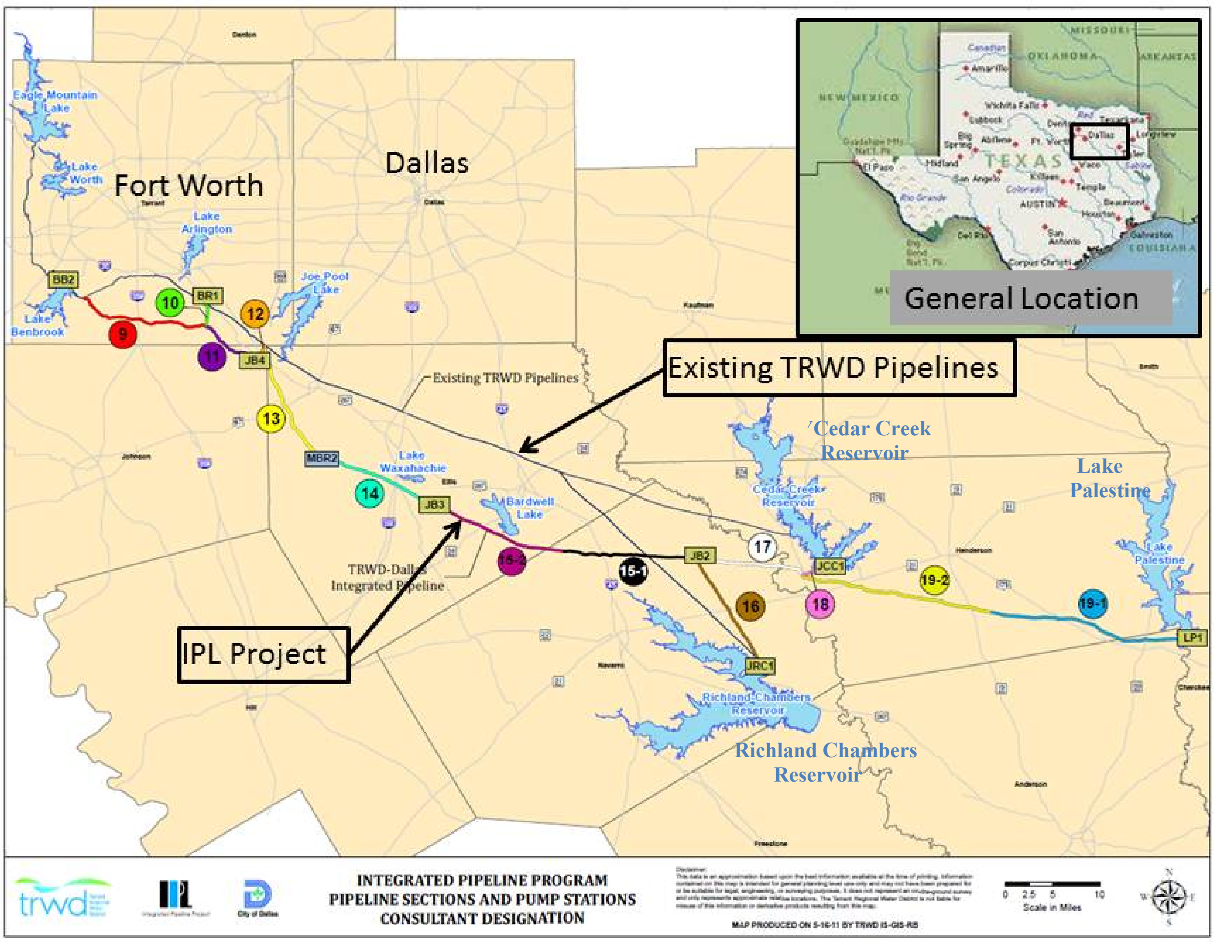

Figure 1 – IPL Project Location

The Tarrant Regional Water District (TRWD) and the City of Dallas Water Utilities currently provide drinking water to an estimated 4.4 million people. Based on developments and updates of City of Dallas and Texas Water Development Board long range water supply planning studies conducted in 2005-2006, it is predicted that population and water demands are likely to double in the next 50 years. The IPL project is developed to provide an additional 350 MGD supply to meet these growing needs.

The IPL Project is being developed in five distinct phases. Phase 1, scheduled for completion in 2018, is currently in construction and includes 70 miles of 84 inch to 108 inch pipeline, a 350 MGD booster pump station including an 80 MG suction reservoir, three facilities interconnecting the new IPL with existing TRWD pipelines, a 450-MG terminal storage balancing reservoir, and ancillary facilities. Phase 2, scheduled for completion in 2020, includes 11.5 miles of 108 inch pipeline, a ½ mile long approximately 14-ft dia. river tunnel, and a 275 MGD lake intake pump station at Cedar Creek Reservoir. Phase 3, scheduled for completion in 2025, includes 12 miles of 96-inch pipeline and an expansion of the TRWD existing Richland Chambers lake pump station to an additional 250 MGD capacity. Phases 4 and 5, scheduled for completion thru 2035, includes the remaining components of the IPL – 50 miles of 84-inch pipeline, 5 miles of 120-inch pipeline in tunnel, two booster pump stations at 200-350 MGD, an 80 MG suction reservoir, and a 150 MGD lake intake pump station at Lake Palestine.

Leadership of TRWD and DWU set out from the beginning to deliver the IPL as a truly sustainable project. Project goals include decision making to balance the "triple-bottom-line" function of "people" (social), "planet" (environmental) and "profit" (economics). Life cycle costing was used to compare various alternates where sustainable payback may be years out from capital outlay. Goals in the planning and design were to provide a reliable system with a 100-year service life. Development of all IPL facilities took into consideration the potential for future expansions and operational scenarios.

For such a large, complex, multi-phase, multi-decade project, the application of Value Engineering was undertaken to provide additional review and evaluation to consider opportunities for economies of scale, to avoid obsolescence in design, to balance flexibility with interconnections to the existing system, to reduce projects risks and to balance concepts of design for the future with current capital costs.

TRWD and DWU selected Robinson Stafford & Rude, Inc. (RSRI) of Gulfport, FL as the Value Management Consultant to conduct a series of Value Engineering Workshops at various design stages. RSRI was responsible for assembly of a team of national experts with various pertinent disciplines. The RSRI team along with various Owner and IPL team members constituted the VE Team. Workshops were held over a period of three years considering: the Concept Design, the 30% design of the joint booster pump stations and suction reservoirs, the 30% design of the lake intake pump stations, the 60% design of the JB3 Booster Pump Station, JB3 suction reservoir and Midlothian Balancing Reservoir (MBR), the 60% design of the Joint Cedar Creek (JCC1) Lake Intake Pump Station, and the 60% design of the various section pipelines. Further, IPL program management team provided informal VE with design reviews on each deliverable and worked collaboratively with Designers to achieve greater value in the projects. The efforts of formal VE and informal VE resulted in an estimated opportunity of savings ranging from 1% of reviewed cost to almost 50% of individual component project costs.

VALUE ENGINEERING PROCESS

SAVE International™ notes that Value Engineering is "a systematic process used by a multidisciplinary team to improve the value of a project through the analysis of its functions." Noting that value of a given component can be defined as the ratio of function to cost, it becomes apparent that the value of a project's design can be improved in three basic ways:

1. Maintaining the result of a design function at a lower cost,

2. Improving the result of a design function at the same or similar cost, or

3. Improving the result of a design function at a reasonable / acceptable increase in cost.

Formal Value Engineering for the IPL Project utilized a consistent process including Pre-workshop activities, multi-day workshop, and post workshop decision making. Pre-workshop activities generally included review of project information (studies, reports, design drawings, specifications, and schedules) and developing cost models. The workshop followed the six phase "Job Plan" as outlined by SAVE International™:

1. Information Phase – Owner and Designer present to VE Team project background, establish VE study constraints, determine economic data for life cycle cost, and define functional requirements.

2. Functional Analysis Phase – Confirm project objectives, determine key items / elements of project.

3. Creative Phase – Generation of large quantity of ideas which could add value to project regardless of idea feasibility (brainstorming session).

4. Evaluation Phase – Select from ideas generated in Creative Phase those with the most merit for further development as recommendations or as a design suggestion.

   Mid Workshop Review – Owner and Designer review ideas selected in Evaluation Phase to determine fatal flaw of ideas that were infeasible OR encourage further development of ideas from Creative Phase that did not make it to the Evaluation Phase selection. Refinement of Evaluation Phase idea list based on Mid Workshop Review.

5. Development Phase – Each idea from refined Evaluation Phase is developed into a narrative description, life cycle cost analysis and comparison of advantages / disadvantages.

6. Presentation Phase – Final day of VE Workshop, VE Team presentation to Owner and Designer

The Post Workshop activities included VE Team preparing preliminary report of findings from the workshop, Designer response to each idea presented from Workshop Development Phase and a decision making meeting with the VM Consultant, Owner/IPL Program staff and Designers.

Utilizing the above noted process resulted in successful workshops. Besides the potential cost savings to be realized from the suggested revisions, certain ideas were presented noting other benefits including:

a. Improved operations / maintainability
b. Improved coordination / constructability
c. Improved consistency of quality control
d. Longer life project elements
e. Reduction in carbon footprint
f. Improved safety

In the section below on IPL Reservoirs design, specifics on some idea development from the formal VE will be presented in more detail. Summary outcomes of the six formal value engineering workshops are summarized in Table 1 below:

Table 1 – IPL Formal Value Engineering Summary

Workshop Topic/Stage	Number of Ideas for Evaluation			Est. VE Savings	Date of VE
	Creative Phase	Evaluation Phase	Accepted / Further Study		
Concept Design	183	49	44	8%	Sept. 2010
30% Booster Pump Stations (JB2, JB3, JB4)	288	53	46	38%	May 2012
30% Lake Pump Stations (LP1, JCC1, JRC1)	285	47	24	16%	Oct. 2012
60% JB3 and MBR	317	56	30	5%	Sept. 2013
60% JCC1	272	59	18	~1%	Oct. 2013
60% Pipelines	209	44	30	4%	Oct. 2013

INFORMAL VE MEASURES

Along with the formal VE, a considerable effort was made in constant refinement of project design criteria and design reviews. This informal VE process was truly a collaborative effort with idea initiation and vetting from amongst staff from Owner, IPL program team, Designer, and program wide service consultants. While not conducted using the six phase process as described above, the savings can be considered a value to project as it reduces cost without compromising the functionality of design. Table 2 lists a handful of the value increase ideas.

Table 2 – IPL Informal Value Engineering Summary		
Category	Description	Est. VE Savings
Crossing Permits	Design of several road and creek crossings were revised from tunnel crossings to open cut crossings	Five crossings saving approximately 2400 feet of tunneling work and 10 shafts
Tunnel Design	Design of several tunnels revised to standard pipe wall and cover depths based on site specific conditions	Reduced shaft excavation by 60,000 cy; reduced pipe materials by 7,500 tons
Vertical pump can	Design revised to allow use of partial can in lake intake pump station	Significant savings in material and installation costs
Steel Coils for Pipe	Purchasing decision allows for early payment for steel coil – assumed savings based on reduced interest costs	Cash flow cost savings
Pipe Wall	Pipe wall design based on pressure class with sidewall support developed by improved embedment materials. Allowing use of 46ksi yield steel for pressure class design.	Approximately 15,000 tons of pipe material
MBR Yard Pipe	Design reconfiguration of MBR site reduced quantity of valves and large diameter pipe	Approximately 500 tons of pipe materials, 2 large dia control valves and vaults
Granular Embedment	Design revision to allow use of local source sand and gravel embedment	Approximate 700,000 tons of embedment material – eliminates 1,000 miles hauling
Reservoir	See below discussion	See below
TOTAL	Informal VE lead to a savings of approximately 4% of project budget	

IPL RESERVOIRS

One of the major components of the IPL system is a group of earthen embankment suction and balancing reservoirs located along the pipeline route. The Reservoir Design Engineer Team, consisting of prime consultant Freese and Nichols, Inc. of Fort Worth Texas and subconsultant Nathan D. Maier, Inc. of Dallas, TX, are responsible for the planning, design and construction of three reservoirs for the IPL. Two of the reservoirs, JB2R and JB3R, are suction reservoirs at the system booster pump stations – design of these reservoirs consists of two 40-MG cells with space for future expansion to an ultimate buildout of four 40-MG cells. The initial buildout of JB3R is scheduled for completion in summer 2015 – JB2R is scheduled for Phase 4 of the IPL Project. The other reservoir, MBR, is a balancing storage reservoir at the system high point – sitting at interface of the 108-inch dia. Pipelines for Sections 13 and 14. MBR is currently under construction and is configured for three approximately 150-MG cells (See Figure 2).

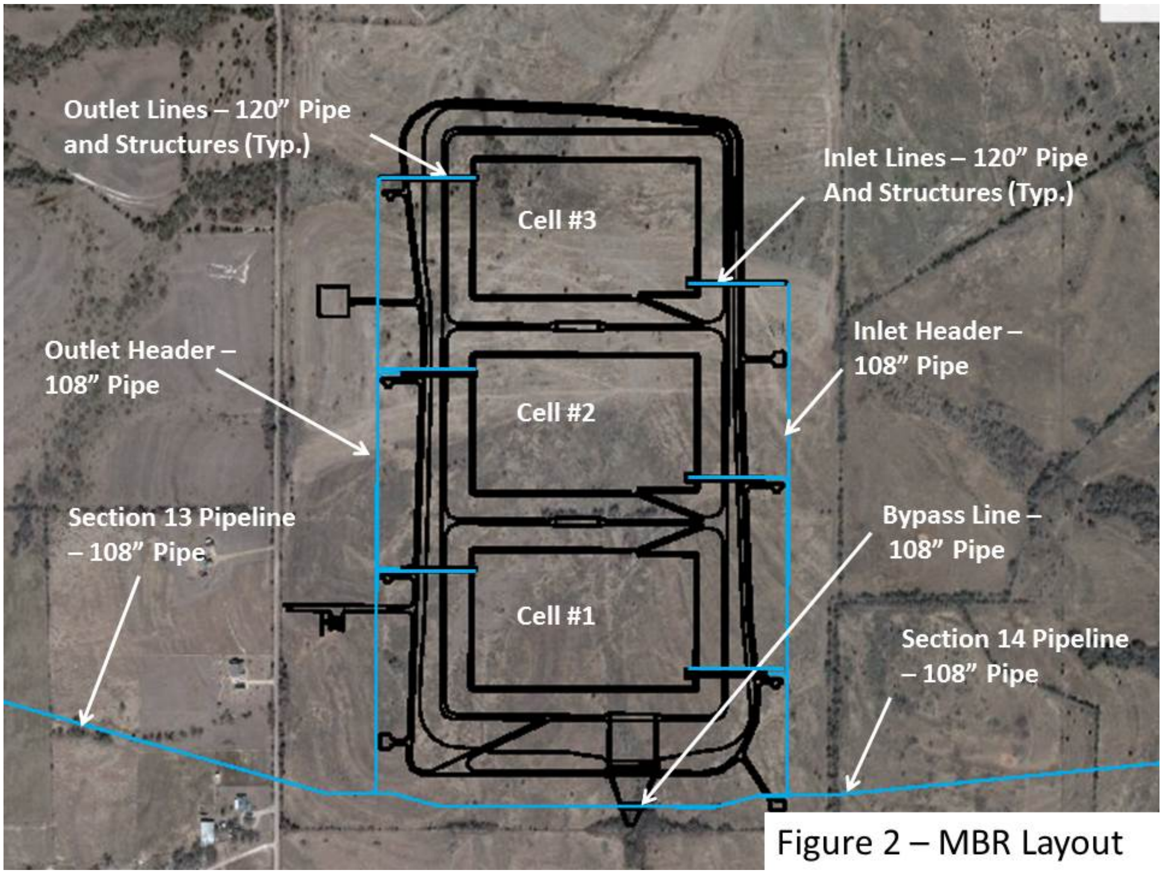

Figure 2 – MBR Layout

The balancing and suction reservoirs located along the proposed pipeline route serve several purposes. At the booster pump stations, they allow for sufficient water to be stored such that the net positive suction head requirements of the pumps are met. They also provide a level of surge control along the pipeline so that, if a valve is closed unintentionally, the surge wave will not travel the entire length of the pipeline, but will likely be contained in a reservoir. The balancing reservoirs will also potentially allow for time-of-day pumping - which is a situation where water is pumped from the supply reservoirs into the balancing reservoirs at a time of day when the cost of pumping is lower. Water can then be gravity fed to downstream distribution points during periods when pumping costs are significantly higher.

Design of IPL reservoirs are also configured with consideration for future expansion and regular operation maintenance. The reservoirs are designed for multiple cells – this allows for some redundancy and maintainability. Each cell at the booster pump station sites contains sufficient volume to backfill the system following a line break / emergency dewatering. The cells are designed to remain serviceable when one cell is down for maintenance and the other is at maximum water height. Similarly, at the MBR, the third cell allows for greater flexibility in operation and maintenance.

Design elements

The basic design of the reservoirs include a). earthen embankments either full clay or with a zoned clay core embankment, b). an interior liner system consisting of a 60-mil HDPE liner over composite geonet and covered with a 9” layer of soil cement, c). exterior slope protection with locally common Bermuda grasses, d). an underdrain system, e). inlet and outlet pipes and structures, and f). an overflow structure. (Figure 3)

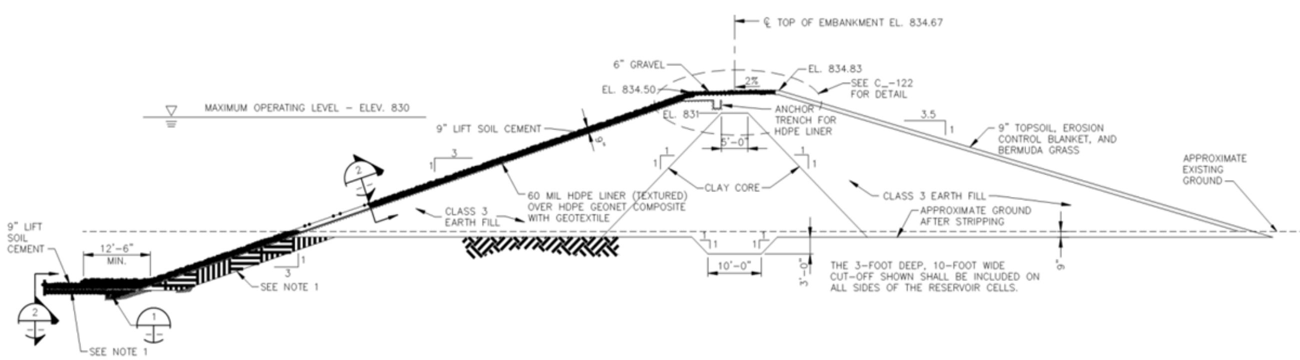

Figure 3 – Typical Design Section

Reservoir Lining - Liners are needed to provide protection from erosion from wave action as well as some additional protection from seepage. In addition a liner provides a means to allow sediment removal from the reservoir. The use of a thin soil cement lining system provides a durable surface which is also economical. Placing the soil cement over the HDPE liner provides a durable relatively impervious surface. (See Figure 4 below)

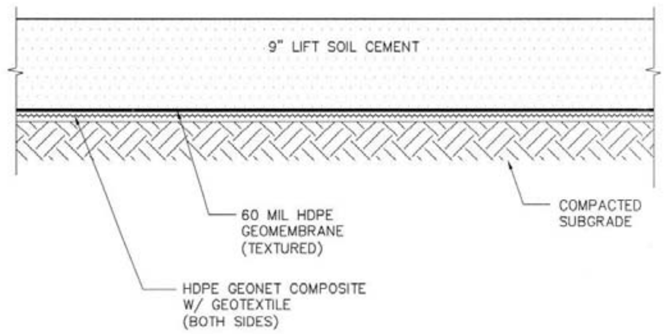

Figure 4 – Typical Liner Section

Inlet and Outlet: The inlet and outlet of the reservoir is designed for a maximum capacity of the pipeline. The inlet transitions from the pipeline through a control valve. The pipeline section which goes under or through the embankment includes a concrete encasement around the pipe. The entrance into the reservoir is through a concrete headwall structure at the bottom of the reservoir cell. The outlet pipe is constructed in a similar fashion with a headwall and concrete encasement around the pipe. The outlet has a trash rack installed to prevent introducing any foreign materials into the pipeline and/or pump station suction header. The inlet and outlet are located on opposite sides of the reservoir to create as much turnover of the water as possible. For JB2R and JB3R – the outlet pipe is sized at 114-inch while the inlet is 108-inch diameter. For the MBR, both inlet and outlet pipe are sized at 120-inch diameter. (See Figure 5 below)

Figure 5 - Construction of Encasement on 108-inch Steel Inlet Pipe at JB3R

Overflow Weir - The reservoir needs an overflow weir to prevent potential overtopping of the embankment. It provides an uncontrolled outlet in case of operator error or lost communication to a pump station or outlet structure. It also provides a discharge capability in the during a significant storm event. The overflow design for all IPL reservoirs includes a broadcrested weir located on the side of the reservoir which will allow flow to move down a protected section away from the toe of the embankment.

VALUE ENGINEERING ELEMENTS

As discussed above, an extensive Value Engineering (VE) review was conducted by multiple consultants on behalf of the IPL Team. The VE approach evaluated several project recommendations in an effort to reduce the cost of the project, as initially proposed, or to provide concepts that added value to the overall project. The VE evaluation looked at all recommendations of the overall IPL Project, which included the booster pump stations and suction reservoirs. Many VE recommendations were considered and discarded, and other recommendations were selected for additional review and consideration. Through a series of meetings, the VE recommendations were further evaluated and selected recommendations were identified to carry into the preliminary design phase and for modification to the draft Preliminary Design Report.

Formal VE recommendations included one major revision coordinating design assumptions at the booster pump station which allowed for reduced operating levels conducive to site earthwork balance. Recommendations also included revisions to steepen interior slopes in the reservoirs and to provide reduced size of clay core at the MBR zoned embankment section.

Informal VE process also led to several revisions which added value to the project. The MBR site was re-designed from four cells at 100 MG each to a three cell at 150 MG each configuration. This allowed greater capacity in individual cells, provided full build out rather than expansion in the future, allows additional capacity for system operations and reduced the overall volume of earthen embankments. Another informal revision included removal of chemical feed from within reservoir outlet structure – which was determined as non-essential following a physical modeling study conducted for the JB3 pump station / reservoir system. As part of TRWD innovative approach to developing technology in design, manufacturing and construction, the JB3R 114-inch diameter outlet pipes were allowed on a trial basis to be cement mortar lined in the plant; which turned out to be a successful technique and provides value to the future projects in the system utilizing this size pipe.

Table 3 provides a summary of the “value” of the VE Process with respect to the reservoir design. The “value” as defined above, i.e. ration of functional unit to cost, for the reservoir is gallons storage per dollar capital construction (gal / $).

Table 3 – Progression of Value Add to IPL Reservoir Projects				
Milestone	JB2 Reservoir	JB3 Reservoir	MBR	Value
Concept Design[1] Jul. 2010	40 MG - $11,046,000	40 MG - $11,046,000	NIC	3.62 gal / $ budget ($276,000 / MG)
IPL Baseline[2] Apr. 2011	40 MG - $11,247,000	40 MG - $11,247,000	200 MG - $48,300,000	3.96 gal / $ budget ($252,000 / MG)
Program Update[3] Mar. 2012	120 MG - $41,650,000	120 MG - $57,746,000	300 MG - $52,003,000	3.57 gal / $ budget ($280,000 / MG)
30% VE Study[4] Dec. 2012	80 MG - $16,454,000	80 MG - $16,580,000	400 MG - $56,952,000	6.22 gal / $ budget ($161,000 / MG)
MBR Reconfiguration[5] Mar. 2013	80 MG - $16,454,000	80 MG - $16,580,000	450 MG - $49,423,000	7.40 gal / $ budget ($135,000 / MG)
60% VE Study[6] Nov. 2013	80 MG - $13,898,000	80 MG - $12,625,000	450 MG - $44,601,000	8.58 gal / $ budget ($117,000 / MG)
Reservoir Phase 1 Bid[7] Aug. 2014	80 MG - $13,898,000	80 MG - $11,387,000	450 MG - $44,223,000	8.78 gal / $ budget ($114,000 / MG)

1. Baseline system from Concept Design. Did not include MBR in Concept Project.
2. System configuration established with MBR included. Minimum sizes of reservoirs for IPL established.
3. System pumping requirements set at much high elevations than anticipated; additional capacity requested requires build out of three cells per reservoir.
4. VE recommendation accepted resulting in lower NPSH elevation and a balanced earthwork site; additional capacity requested at MBR
5. Informal VE results in reconfiguration of MBR from 4 cells to 3 cells
6. Formal VE results in steeper interior slopes; informal VE results in removal of 24" outlet and chemical feed revision
7. Actual bid results from JB3R and MBR – MBR combined project with adjacent pipeline; JB3R bid in advance of JB3 Pump Station for coordination ease.

As can be seen, the IPL Value Engineering process both formal and informal provide great value to the project and for the reservoir component resulted in more than doubling the value during the life of the design process.

Triple Bottom-Line Assessment of Alternatives for a Large-Diameter Transmission Main from a Congested 280-MGD Water Treatment Plant Site

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Abstract

Due to a series of high impact failures of prestressed concrete cylinder pipe (PCCP) in its water distribution system, the Washington Suburban Sanitary Commission (WSSC) has developed an aggressive program of field condition assessment to identify pipes in danger of failure. However, it found that it was not able to apply these techniques to the two parallel PCCP pipes located within its primary water treatment facility, the Potomac Water Filtration Plant, because the flow demands would not permit the existing 78-inch line to be removed from service for condition assessment. A Business Case evaluation was convened to develop and compare alternatives for installing a full capacity redundant transmission line to carry finished water from the plant, which was complicated by the severely congested nature of the facility, which had grown and been added to multiple times in its history. The alternatives were evaluated using Triple Bottom Line techniques for cost, societal and environmental impacts. Alternatives considered included a tunnel, multiple near surface alignments, and pump station reconfigurations, plus the "No Action" and "Status Quo" options.

BACKGROUND INFORMATION AND PROBLEM DEFINITION

Due to adverse experiences with precipitous failures of prestressed cylinder concrete pipe (PCCP) the Washington Suburban Sanitary Commission (WSSC) has developed a proactive strategy to detect imminent failures in these pipeline assets prior to failure. (Marshall, 2009) This program entails an internal inspection of the targeted pipeline assets, those 48-in and larger, on a seven-year cycle and installation of an acoustic fiber optic (AFO) monitoring system for the purpose of detecting telltale wire breaks that are indicative of probable failure. The internal inspection and the installation of the permanent AFO monitoring system require the pipeline to be taken out of service for a period of time.

In developing the implementation plan for the inspection program it was found that there was a problem in scheduling the outage for certain finished water transmission lines from the utility's principal water treatment plant. The Potomac Water Filtration Plant (WFP), located in Potomac, MD, provides treated water to more than a million people in Montgomery and Prince George's Counties. The current hydraulic treatment capacity of the plant is 288 MGD. Finished water is conveyed via two onsite pumping stations to two principal pressure zones in the water distribution system. The Main Zone Pumping Station (MZPS) discharges through two parallel PCCP lines to the Main Zone transmission line network at a point just outside the plant premises, with the larger M2/78-in line serving as the primary supply and the smaller M1/48-in line serving as a supplemental backup.

The larger M2/78-in line normally carries the entire flow, with the M1/48-in normally not in service. In reviewing the operations of the two mains it was determined that the capacity of the M1/48-in line was not sufficient to carry the entire flow, and in fact the larger pipeline had not been taken down for inspection or service since its original construction. On the basis of this scheduling exercise it was realized that this lack of redundancy not only interfered with this critical pipeline inspection program, but also constituted a serious operational resiliency issue.

A project was initiated to design a third transmission main to serve the Main Zone Pumping Station; however, it was realized that this would be a difficult task because of the congested condition of the plant site, with numerous large water mains and plant utilities sharing the narrow corridors between plant buildings. (See Figure 1) In order to rigorously compare alternative approaches for providing the desired redundant capability, and to rigorously justify the expense that would be required, WSSC initiated a business case analysis of the issue.

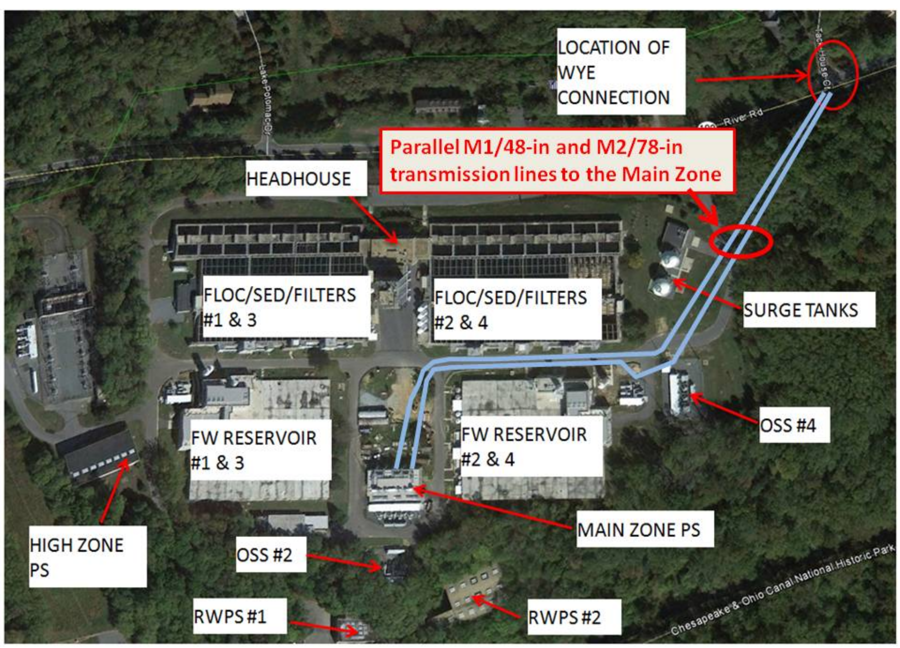

Figure 1. Potomac Water Filtration Plant site plan

BUSINESS CASE ANALYSIS FRAMEWORK

WSSC has developed an Asset Management Program which incorporates elements defined in the Water Environment Research Foundation's SIMPLE program (WERF, SAM1R06i) and the Environmental Protection Agency's Sustainable Infrastructure program (EPA, 2012). One element of this program is the utilization of a defined set of business case analysis tools for the evaluation of alternatives for implementation in capital and operations initiatives. The objective is to evaluate projects and alternatives on comparable metrics, using a triple bottom line analysis of financial, environmental and societal impacts. Three spreadsheet based algorithms are used to accomplish a consistent monetization conversion, as listed below and in Figure 2:

- the Risk Reduction Tool, which is used to estimate the probability of failure of an asset, and the consequence of failure, to result in a quantified risk;
- the Life Cycle Cost Tool, which calculates present worth costs of alternatives, and the internal benefits that would accrue to the utility; and
- the Benefit Cost Tool, which is used to extend the benefit calculation where it is determined that there are significant benefits (environmental and societal) that accrue external to the utility.

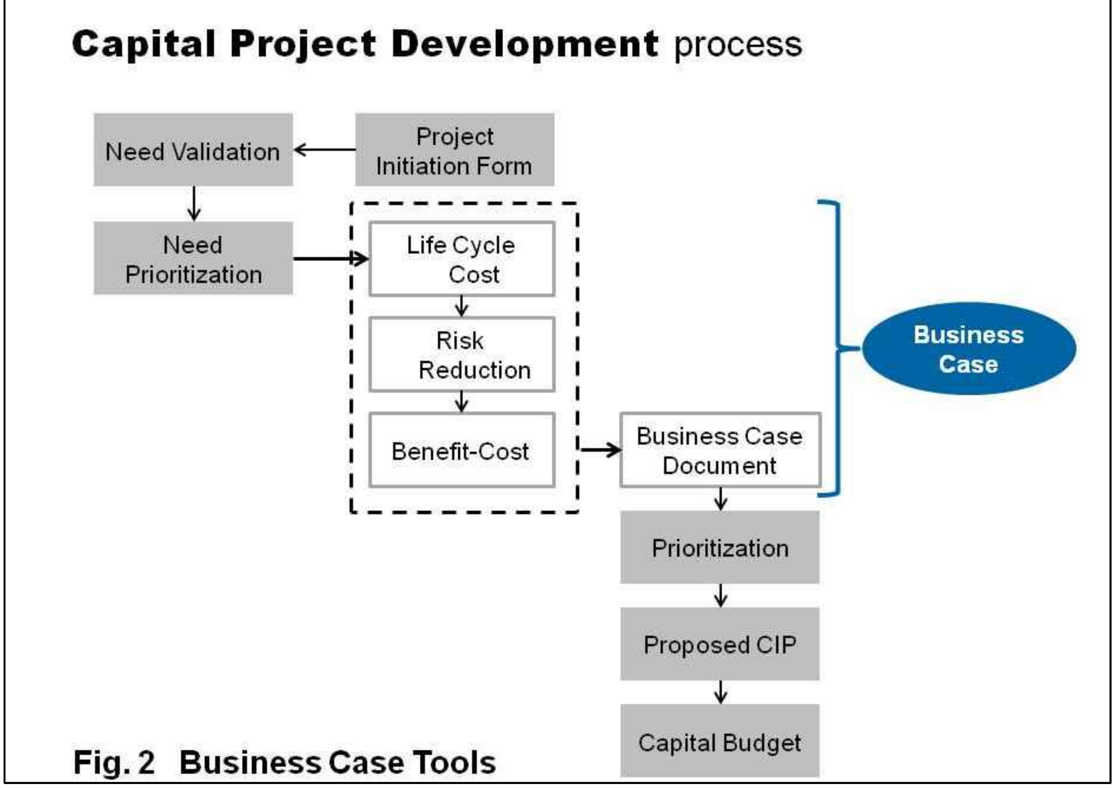

Fig. 2 Business Case Tools

ALTERNATIVES DEVELOPMENT

After a preliminary screening of alternatives that by definition would not meet the technical standards requirements of WSSC, or that represented only unacceptable partial solutions to the defined problem, a set of final alternatives were developed for final analysis. Essentially, the alternatives consisted of two “de facto” options included in every business case analysis, plus a suite of options for routing a redundant transmission main through the congested layout of the treatment plant.

- No Action Alternative - This is a “run to failure” option. It involves taking no action, either capital project or continued operational activities to forestall failure. It is clearly unacceptable, but is used in the analysis as the benchmark risk that is to be mitigated by the other alternatives.

- Status Quo Alternative - This is generally as the name implies the continuation of current operations and maintenance activities to prevent failure. For the purpose of this analysis it was somewhat modified to include the implementation of some additional action that had been developed in this project, but which would not entail a significant capital expense. In this case, it entailed the purchase of a repair kit to have in stock in the event of a failure of the M2/78-in pipeline. (This pipeline is the only reach of PCCP pipe in the WSSC inventory of this size, so it had not been previously stocked.)

The routing options for a new redundant pipeline are depicted in Figure 3, and listed below in clockwise progression around the plant site.

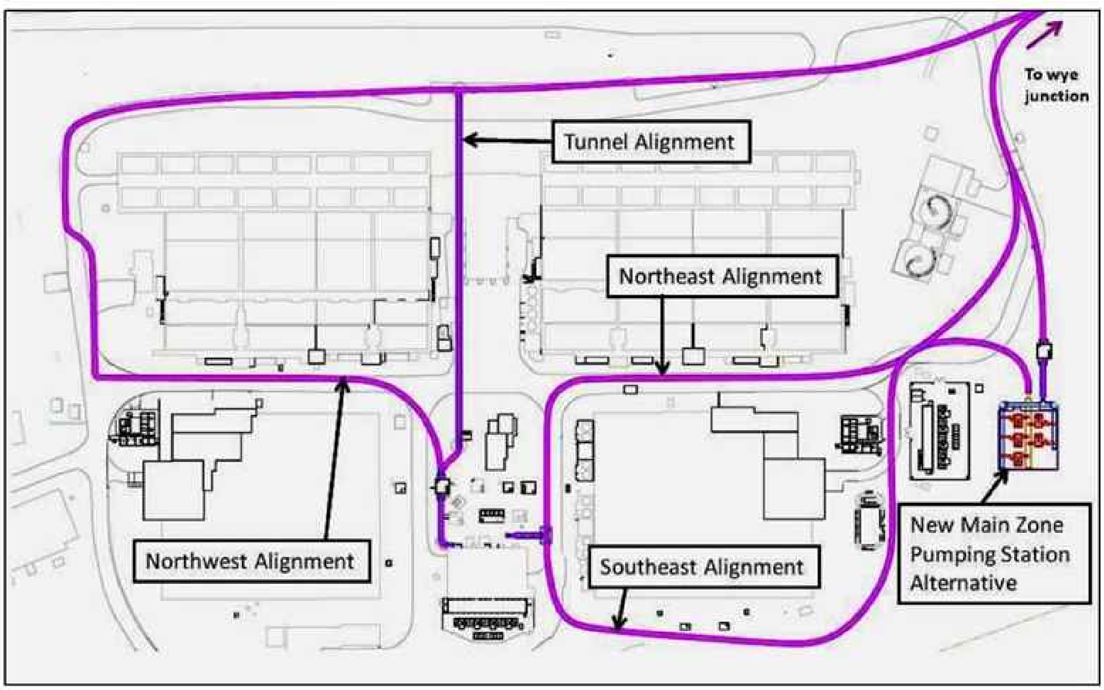

Figure 3 – Alternative Routes for Redundant Pipeline

- Northeast Alignment - This alternative consisted of replacement of the current inadequately sized M1/48-in line with a new 84-in steel main in the same congested corridor with the M2/78-in line. The disadvantages of this alternative were that the construction of the new main would be so close to the existing primary main that there would be significant risk during construction of putting the entire Main Zone out of service in case of accidental damage. The alignment also entailed the crossing of several critical chemical feed lines and other utilities, and would be in very close proximity to critical process tankage.

This construction risk of outage to adjacent assets (i.e. the parallel water transmission mains, the eastern floc/sed basins, and the finished water reservoirs), with the consequent water service outage to the customers in the Main Zone was found to be the dominant factor in the triple-bottom line assessment of this alternative.

- Southeast Alignment - This alternative consisted of a new 84-in steel main routed from the Main Zone Pumping Station to the southeast around the finished water reservoirs #2 and #4 and then north out of the plant site to the target junction. This option has the advantage that it avoids the most congested corridors of the plant, although it still does cross some utilities. However, it had significant disadvantages in that it would have to be installed on sloped terrain, and would involve removal of a significant number of trees within the viewshed of the National Park Service's C&O Canal. This latter consideration suggested that obtaining needed permits for construction would likely take significantly longer than other alternatives.

  This environmental impact was the principal factor in the triple bottom line analysis of this alternative, plus a calculated delay in the risk-reduction benefit that would accrue due to the likely delay in project implementation because of uncertainty of how long it would take to receive National Park Service concurrence (if ever.)

- Northwest Alignment - In this alternative routing the new 84-in pipeline would follow the alignment of an existing, abandoned 36-in pipeline in the corridor between the floc/sed basins #1 and #3 and finished water reservoirs #1 and #3, and then north out of the plant to the main roadway and eastward on to the target junction point. This alignment also entailed crossing of significant plant pipelines and utilities, and was the longest alignment.

  The triple bottom line analysis considerations for this alignment were similar to that of the Northeast Alignment (i.e. significant construction risk which could result in water service outage to a large population.)

- Tunnel Alignment – This option was defined as a rock tunnel routed from the Main Zone Pumping Station under the head house building to the north out of the plant site and then east to the target junction point. The alignment had among the shorter routes, and was estimated to have the least cost of the pipeline alignments due to the avoidance of potential construction adversities involving at-risk pipelines and other plant assets.

  The principle factors favoring this alternative in the triple bottom line assessment was that it was seen to entail minimal environmental impact, and the least construction risk that could result in large scale water service outage.

- <u>New Main Zone Pumping Station</u> - The last alternative considered was construction of a new Main Zone Pumping Station on the east side of the plant. It was anticipated that construction of a whole new pumping station would be the most expensive first cost option, but it was hoped that there might be feasible routings for the pump station feed lines that could avoid the routings of the discharge main in the other options. However, this was not found to be the case, so the construction risks of the other options also applied to this alternative.

  The much larger construction cost, coupled with the less-than-hoped-for construction risk reduction, factored heavily against this alternative in the triple bottom line assessment.

ANALYSIS

The business case tools were applied to the alternatives listed above, including calculation of present worth construction and operating cost of each option, and the monetized assessment of adverse environmental impacts and risks of failure. It was found that the assessed risks, primarily potential extended large scale water system outages due to construction accident were the primary drivers. The monetized comparisons are depicted in Tables 1 and 2 below.

Table 1 – Present Value of Total Costs and Annuitized Cost Streams

Alternative	*Analysis Period*	*Near-term Anticipated Capital Cost**	*Present Worth Total Costs**	*Annuitized Cost Stream**
No Action	139 yrs	-	$18,609	$748
Status Quo	139 yrs	-	$19,200	$771
Northeast Alignment	110 yrs	$23,400	$22,775	$923
Southeast Alignment	112 yrs	$24,980	$25,497	$1,033
Northwest Alignment	110 yrs	$30,590	$29,571	$1,199
Tunnel Alignment	110 yrs	$23,120	$22,511	$913
New Main Zone PS	110 yrs	$49,720	$42,075	$1,706

** Monetary values are in thousands*

Table 2 – Benefit-Cost Ratios of Alternatives

Option	*Present Worth of Benefits (thousands)*				*Present Worth of Costs (thousands)*			B-C Ratio
	Financial & Social	*Environ -mental*	*Const Risk Adjust*	*Total*	*Financial*	*Social & Environ -mental*	*Total*	
No Action	$0	$24	$0	$24	$18,609	$50	$18,659	0.00
Status Quo	$74,322	$35	$0	$74,357	$19,200	$0	$19,200	3.97
NE Alignmt	$110,262	$91	($2,963)	$107,390	$22,775	$0	$22,775	4.72
SE Alignmt	$110,263	$85	($1,017)	$109,331	$25,497	$4,488	$29,985	3.65
NW Alignmt	$110,263	$91	($1,017)	$109,337	$29,571	$0	$29,571	3.70
Tunnel Alignmt	$110,263	$91	$0	$110,354	$22,511	$0	$22,511	4.90
New Pump Stn	$110,262	$1,220	($2,963)	$108,519	$42,075	$0	$42,075	2.58

* *Monetary values are in thousands*

Since completion of the business case analysis, the recommended tunnel alternative has been moved forward into design. As of this writing (in April, 2015) the design criteria for the project is being finalized, with 30% design expected by the end of 2015, and anticipated procurement bidding by the summer of 2017. At this stage in the design, the planned tunnel alignment has been modified from the due north alignment configured in the business case, to a somewhat more northeasterly route, reducing the total pipeline length.

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Value Engineering of Conveyance System Projects on a Large Wet Weather Program

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Abstract

The City of Baton Rouge/Parish of East Baton Rouge (C-P), Louisiana has designed and is implementing a sanitary sewer overflow (SSO) abatement program that involves conducting comprehensive rehabilitation and capacity improvements to the collection system as part of an EPA/DOJ Federal Consent Decree. The C-P developed the Program Delivery Plan (PDP), with a current estimated cost of $1.6 billion. In order to manage costs, value engineering (VE) has continuously been a critical component of the Program. Initially, projects in design that had a construction cost estimate of $25M or greater underwent VE. The savings realized from this VE effort was approximately $93M, with the largest savings at the South WWTP ($85M). For all remaining conveyance system projects in design, VE proposals were made by the Program management team (PMT) that could be generally applied. The VE proposals, resulting in $25M in savings, typically involved reduction in scope such as consolidation of buildings, deletion of valve vaults at pump stations, and changes to conduit materials. In order to execute projects to immediately alleviate SSOs in the system, the initial phases of implementation involved making decisions with the limited information available at the time. As components of projects were brought online, a VE committee comprised of staff from the PMT and the C-P was formed to challenge the initial design criteria and evaluate the impact of projects to ensure that each project was delivering the highest value for its construction cost. The VE committee used additional system information, as well as lessons learned from the initial projects. The committee reviewed the proposed pump station capacities and pipeline segment sizes as individual components and their function in the system, in relation to the existing infrastructure and anticipated future development. If a planned pump station or pipeline segment upgrade was questioned, a VE review of the C-P's hydraulic model was requested. If the future system model showed that the existing pump station or pipeline segment had no SSOs, and the C-P advised that the portion of the system did not have a history of SSOs, then the pump station or pipeline segment was removed from the project. Other examples of proposals include technical review to ensure that a pump station and its associated force main will operate at an optimal point on the pump curve and modifications at pump stations

(e.g. lower wet well inverts to allow for future tie-ins). This VE review is ongoing and has resulted in over $20.5M in savings as of February 2015. Many improvements in the system's future operational ability have also been made. Through VE reviews, the C-P has captured over $138M in savings as of February 2015. This paper will present a variety of specific examples of technical modifications that were made within the SSO Program, in order to achieve the Program's overall goal through the ever-changing economic climate, in a method that could be utilized on other large wet weather programs.

INTRODUCTION

As mentioned above, the C-P has developed and is implementing a sanitary sewer overflow (SSO) abatement program that involves conducting comprehensive rehabilitation and capacity improvements to the collection system as part of an EPA/DOJ Federal Consent Decree. The C-P entered into the consent decree in 2002. As a response to the consent decree, the C-P wanted to implement an affordable, constructible and sustainable SSO Program that addresses present challenges while planning for future growth. The C-P desired a SSO Program that would be implemented with true partnership and full accountability to the public. This strong partnership between the C-P and the SSO Program management team (C-P/PM Team) has been instrumental to the success of the SSO Program and has been critical to the effective implementation of a value-focused mentality through its implementation.

The SSO Program formally began in 2007 and included the following key goals:

- Reduce excess wet weather flows that cause SSOs
- Rehabilitate the collection system
- Increase the hydraulic capacity of the collection system
- Accommodate growth in project areas
- Comply with wastewater treatment plant National Pollutant Discharge Elimination System (NPDES) permit
- Comply with the terms of the Consent Decree

PURPOSE OF VALUE ENGINEERING

As large consent decree driven programs are developed, there should be a constant emphasis to balance the success factors of time, cost, and quality (Figure 1). Value Engineering (VE) can be used as a tool to maintain an appropriate balance throughout the life of the program. The impact of VE can reduce time, save money, or add quality to the overall program.

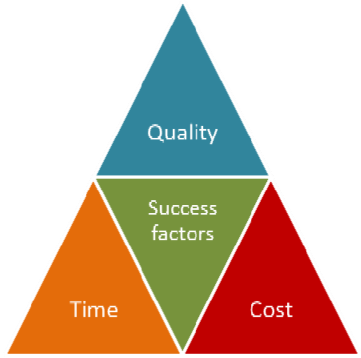

Figure 1: Project Management Success Factors[1]

In order to execute projects to immediately alleviate SSOs in the system, the initial phases of implementation involved making decisions with the limited information available at the time. The Program Delivery Plan (PDP) was developed by the C-P/PM Team in 2007, and implementation began almost immediately thereafter. Due to the nature of the consent decree requiring immediate action, the SSO Program initially had a strong emphasis on time, in order to provide documented progress toward consent decree milestones. The goal during this phase was to begin implementation and gain momentum and progress toward the federally mandated schedule requirements.

After the momentum was established, the C-P/PM Team began to rebalance the focus with cost and quality, through VE. A series of VE phases occurred over the following years, and the value-focused mindset of the SSO Program began. The C-P/PM Team adopted the value-focused mindset to ensure they were achieving the highest value for the investment, and to be good stewards of the taxpayer's dollars. While there were many large impact VE ideas implemented, the value-focused mindset was instilled in the C-P/PM Team, impacting daily decision making such as recommendation of construction materials on a project site, analysis of routes on small diameter force main alignments, and everything in-between. The primary phases of VE, which occurred in series, are the following:

1. LARGE PROJECT VE REVIEW: Performed on projects with a construction cost estimate of $25M of more
2. OVERARCHING VE REVIEW: High-level review performed on remaining projects which allowed for across-the-board implementation of ideas
3. DETAILED CAPACITY PROJECT VE REVIEW: Detailed VE review performed on remaining capacity projects that allowed for project specific ideas to be implemented
4. ONGOING VE EFFORTS

These VE phases were performed by a combination of C-P staff and technical PM team members. This allowed for representation and involvement from C-P stakeholders, such as collection system maintenance, engineering, plant management, and finance, ensuring that an informed decision was being made that would meet the future needs of all departments. Over the course of the SSO Program implementation, personnel changes naturally occurred, which allowed for fresh eyes and perspectives in the process. The total estimated savings from all phases to date is $135M.

LARGE PROJECT VALUE ENGINEERING REVIEW

VE was performed early in the program on projects with a construction cost estimate of $25M or more, since it was felt that focusing on the higher cost projects would get the most "bang for the buck" in VE.

Several projects were included in this large project VE, including the three projects at the South Wastewater Treatment Plant (SWWTP), the three storage facility projects, two large pump station projects, and the pipeline projects as a whole. The C-P/PM Team generated a large list of ideas, shown below in Table 1, most of which were agreed to and implemented.

Table 1

Large Project VE Review Summary

VE Concept	**Accepted**
Enacting a sales tax exemption on materials for all projects	YES
Changing the design of the trickling filter recycle pump station at the SWWTP	YES
Eliminating a building over the preliminary treatment facility	YES
Changing the type of piling at the SWWTP for the new large facilities	YES
Utilizing an existing chlorine contact basin at the SWWTP instead of constructing a new basin (one new basin was constructed instead of two)	YES
Modifying the design of the solids contact basins at the SWWTP	YES
Eliminating electrical system duplication between two of the SWWTP projects	YES
Changing one of the large pump stations in the collection system from wet pit/dry pit to submersible, thereby also eliminating a building	YES
Eliminating piling and the valve pit cover at one of the other large pump stations in the collection system	YES
Deleting valve vaults at larger (4+ pump) pump stations, unless where necessary due to hydraulic conditions	YES
Reducing use of directional drilling and other trenchless methods for pipeline projects so that they were utilized only when open cut was not an option or was the more expensive option	YES

Changing the conduit material at pump station projects from polyvinyl chloride (PVC) coated rigid galvanized steel (RGS) to Schedule 40 PVC encased in concrete.	NO (C-P experienced previous installation issues)
Pre-purchasing pipe and fittings for pipeline projects	NO (not feasible due to timeframe)

The accepted changes resulted in an approximate savings of $90M in construction cost. It must be noted that the VE stressed that any accepted changes still needed to accomplish the goals of reducing SSOs in the system and treating wet weather flow at the WWTPs. This is best supported by the three phases of the SWWTP projects that are now in service with the final phase recently completing startup. VE savings at the SWWTP are estimated at $85M, but equally impressive is the effluent quality. Effluent biological oxygen demand (BOD) has been reduced by 66%, while TSS has been reduced by 54%. This outcome reemphasizes the value proposition that even if some portions of the projects are removed, a high level of quality can be maintained and successful treatment can still be a result.

Also early on in the SSO Program, the C-P/PM Team made a value-focused decision to construct pump station projects separately from pipeline projects across all projects. This decision was primarily to minimize the contractor's overhead. If the a group of pump stations and their associated force mains were all within one project, the C-P would have paid a markup to a general contractor for something that would be performed by a sub-contractor. The C-P/PM Team targeted the sub-contractors directly, by forming projects with similar construction methods, thereby reducing costs. With this decision, coordination between contractors became a focus, as the projects were sometimes dependent on each other. Even with the coordination efforts and issues that arose, the C-P/PM Team was still getting the benefit from this decision.

OVERARCHING VE REVIEW

The Large Project VE Review was helpful in keeping the costs of the projects near the original budget of the SSO Program. However, during the course of a program, unforeseen challenges occur. The beginning of the SSO Program coincided with the economic downturn. However, as the Program progressed, the economy experienced significant changes and pricing increased dramatically. During this time, a Consent Decree modification was issued by the EPA/DOJ, extending the compliance period by four (4) years but also adding several projects. This revision presented an opportunity for the C-P/PM Team to explore additional VE options. The added projects increased the overall program cost, but also provided time to consider ways to meet the requirements with less costly solutions.

The first portion of this overarching VE review was to form a committee to look for areas where costs could be saved. The committee then presented the VE ideas to the C-P/PM Team to determine which of the proposed ideas would be accepted and carried out. The ideas that were accepted and carried out by the C-P are included in Table 2.

Table 2

Overarching VE Review Summary

VE Concept	**Accepted**
Consolidating several new buildings at various locations into two new buildings that would be part of a new Environmental Services Facility to house the engineering, operations, and maintenance aspects of the Environmental Services Department, which includes sewer	YES
Changing construction materials of the two new buildings to be more economical	YES
Accepting some Contractor-initiated cost proposals at the SWWTP	YES
Deleting a preliminary treatment train at the North Wastewater Treatment Plant (NWWTP), since that train was needed for future flows only	YES
Deleting building improvements at the NWWTP from a capital project and moving them to the operations and maintenance budget	YES
Deleting valve vaults at the duplex and triplex pump stations, unless where necessary due to hydraulic conditions	YES
Replacing PVC-coated RGS conduit with PVC conduit encased in concrete at larger (4+ pump) pump stations	YES
Changing rehabilitation projects to meet a goal of rehabilitating as much pipe as possible while still meeting a 20% reduction in project budget	YES
Deleting the new gravity influent pump station (GIPS) at the NWWTP	NO
Deleting the generators at the NWWTP, utilizing an existing secondary feed instead	NO
Providing portable generators only for 1 out of every 5 duplex pump stations	NO
Utilizing limestone driveways instead of concrete driveways at new pump stations	NO

Pre-purchasing pipe and fittings for pipeline projects	NO
Utilizing SDR 26 instead of SDR 18 piping for 12-inch and under PVC pipe	NO

The VE items that were accepted had an estimated construction value of $45M. As can be seen, some of the items, such as valve vaults and conduit materials, had been either partially accepted or rejected in prior VE sessions, but they were accepted during this VE due to the rising costs of construction. It was determined at this time that the cost savings of these changes outweighed the benefit of the items not changing.

As seen in the above table, the new GIPS at the NWWTP was deemed necessary to meet the goals of the SSO Program. The two generator proposals were rejected, due to the likelihood of power outages during large storms, especially hurricanes, which have previously adversely affected the C-P's ability to convey and treat wastewater. The other three items were rejected because it was felt that the cost savings that could be realized (a total of $4M for the 3 proposals) were not enough to outweigh the benefits that these items had to the projects.

DETAILED CAPACITY PROJECT VE REVIEW

As previously discussed, the C-P/PM Team has had natural turnover, and following the Overarching VE Review, several new members joined the VE committee.

The hydraulic model was the first item challenged by the new committee members. East Baton Rouge Parish doubled in size in the immediate aftermath of Hurricane Katrina in 2005 due to evacuees from the New Orleans area. While many of these people eventually returned home, this event changed the way City leaders planned for the future. The concept of the SSO Program was to design for future flows if a pump station must be replaced due to SSOs. The flows in the model were adjusted based on the population boom following Hurricane Katrina. While it is important to plan for future development, some of the future flows were exaggerated. For example, some areas were completely developed with commercial/light industrial businesses but had significant future flows in the hydraulic model. The VE committee and C-P officials ultimately agreed that the future flows in the model could be reduced. In the unlikely event the area is redeveloped with a significant increase in flows, the sewer capacity issue could be addressed at that time through the capacity reservation process that is already in place.

Challenging the hydraulic model proved to be beneficial because in some cases it allowed rehab of gravity sewer lines in lieu of upgrading and replacing. The reduction in future flows resulted in the gravity line being properly sized. SSOs were alleviated by point repairs and/or CIPP. This concept has resulted in a savings of $7.3M to date.

Another cost saving concept is the extension of force mains in lieu of upgrading gravity lines. Baton Rouge has a flat topography and relies heavily on pump stations in the collection system. Several pump stations discharge into a gravity sewer that ultimately flows to another pump station. In many cases the pump stations are the reason for SSOs in the gravity system. Since the force mains are smaller in diameter and installed at a shallower depth, it was determined to be much more cost effective to extend the force main to the downstream pump station resulting in a savings of $3.8M.

Another item that didn't necessarily save money but added value to the Program was consideration of wet well depth. As mentioned previously, one of the goals of the SSO Program was designing for the future when possible. The thought here is to lower the wet well invert of replacement pump stations so future developments can connect to them via gravity sewer. This will prevent the need for more pump stations within a close proximity in the future. Though it did not save money on this SSO Program, there is a potential for long term savings in reduced O&M costs.

The new committee also focused on analyzing return on investment. For instance, a force main could be upgraded and provide higher capacity, but that increase may not be worth the capital costs. If additional capacity is necessary in the future, other options such as upgrading the pumps could be a more cost effective solution. Therefore, the capacity projects were reviewed with an eye toward the future flows vs. the existing system "real world" SSO issues. Although future flows are important to be addressed, in some areas the growth predicted in the hydraulic model was so far into the future that the return on investment was not great enough. If it was felt that upgrading the force main or upsizing pumps for a future flow that may or may not even reach that pump station, then often the pump station capacity or force main size was reduced from the original plan or left at existing.

VE EFFORTS DURING CONSTRUCTION

As the SSO Program transitioned to a heavy construction phase, the trend of VE items tended to focus on adding value during construction. Sometimes the changes were made as lessons learned from previous construction projects, and other times somewhat expensive changes were made in order to reap the full value of quality materials that were already being purchased.

Throughout the various phases of VE, other items were suggested that didn't necessarily save money. For instance, limestone backfill in streets for the pipeline projects was added, even though this change in backfill material from sand to limestone added construction cost to the pipeline projects. However, the C-P had experienced issues in roadways with pipelines installed with the previous backfill material, so it was determined that it was worthwhile to change the backfill material, even with the extra cost, so that the citizen's money was being spent responsibly. The benefit of an extended roadway design life and reduced road rehabilitation outweighed the cost of the limestone backfill. In addition to limestone backfill in streets, the C-P also implemented several pipe material specifications to make sure that the pipe material used in the projects would last for years to come, such as using SDR 18 PVC pipe instead of SDR 25 PVC pipe.

As mentioned previously, the four year extension of the Consent Decree provided the VE team with an opportunity to reevaluate the remaining projects. That task is still ongoing. In addition to the concepts previously discussed, projects are reviewed with recently constructed projects in mind for additional cost saving measures. For instance, bypass pumping during construction is costly. Revisions to the site plan are considered to reduce the duration of bypass pumping. There is no firm cost savings associated with this effort but it has been a consideration. It also reduces noise during construction which reduces complaints from residents in the area.

Not only did the extension of the Consent Decree allow for a more in depth VE review, it also provided more time for construction. Many of the construction schedules were compressed in the original PDP. The extension allowed the PMT to add contract days to projects that weren't time sensitive. Again there is no firm cost savings but contractors commented positively on the longer duration. These are several ways projects were continuously reviewed based on experience and outcome of previous projects.

LESSONS LEARNED

Consent Decree programs by nature are massive undertakings. All of the dynamic factors that occur will inevitability impact the Program and come into play. The key is anticipating, when possible, and reacting to those factors, in a way that maintains a balance between time, cost and quality.

The following are lessons learned throughout the VE process:

1. VE should begin as soon as possible in the development and implementation of a consent decree program. Even with the requirement of immediate action and schedule compliance, the highest regard for cost and quality should be maintained from the beginning. The work being done on a program always has to be defensible. VE creates a defensible decision making atmosphere in which decisions that were made created value.
2. VE should be continuous, not intermittent. Over the course of a program, there can be periods of time when VE is not a strong focus, and the team eventually has to refocus on the balance. It is far easier to keep the VE mindset and evaluations as an ongoing effort than to restart VE processes with potentially new team members.
3. Stakeholders (groups such as engineering, operations, collections, finance, engineering, plant management) have different views on cost, time and quality, and the balance of the three success factors. A higher level of influence of one stakeholder could impact the balance. A collaborative environment of all stakeholders' interests is critical throughout the life of the program.
4. The VE process needs to continuously focus on the ultimate goals of the program or project. In this case, the VE committee considered ideas for advancement only if they met the goals of mitigating SSOs and improving wet weather collection and treatment.
5. VE ideas need to be reviewed with a focus on whether or not they are technically sound, such as the limestone backfill that was added, even at extra cost.

6. The C-P/ PM Team should continuously evaluating construction methods based on in-field successes or failures and actual costs of installation.
7. Stewardship of the public's money should be the focus of the VE team. Ideas that will provide savings to the public are good. However, those ideas must not create additional O&M costs or future capital costs that will eventually outweigh the original capital savings.

CONCLUSION

Approximately 53% of the projects in the SSO Program have been completed while another 27% are currently in construction. Through VE reviews, the C-P/PM Team has captured over $138M in construction dollar savings as of February 2015. With 66% of the SSO Program complete on a cost basis, the C-P has already seen a reduction in SSOs during wet weather events. Through the series of VE reviews spanning over 7 years, the value-focused mindset has been instilled in the C-P/PM Team, creating a constant satisfaction that the team is achieving the highest value for the capital investment, serving as good stewards of the taxpayer's dollars.

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Tools for Successful Risk Management of Your Next Underground Project

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Abstract

Underground projects for civil infrastructure continue to become more complex. Engineers and contractors are increasingly being asked to design and construct projects that expand the operating envelope in terms of pipe diameters, project lengths, ground conditions and other new factors outside of prior constructed projects. When assessing the complexity of the project, all parties need tools to identify and assess project risks. Risk management should be part of the planning, design and bidding process. It should occur early in the project life cycle to maximize the ability to influence the project's outcome. Frequently the focus of risk management is only on technical risks. Along with technical risks, financial, contractual and third-party risks also need to be assessed. The project team needs to assess who is the best party (e.g., owner, engineer, contractor or insurance provider) for assigning and then managing the risk. Early decisions in a project have the most ability to influence project objectives and cost at a lower investment. As the project progresses, decisions made later in the project life cycle have less ability to influence the final project cost while implementation becomes more expensive. Underground construction is inherently risky. The more risk which is passed onto a contractor through contract documents, the higher the contingency will be during contractor pricing and bidding. This paper will discuss the three major stages of risk management – 1. Risk Identification, 2. Risk Analysis, 3. Risk Management and Mitigation. For every risk, the potential impact of the risk needs to be studied. One can then assess the probability of the risk occurring, the impact of the risk and the urgency of the risk. The combination of these factors can provide a relative risk index for making comparisons between risk items. Techniques used for evaluating, quantifying and comparing risks will be presented. Specific examples will be shared from past risk management programs related to pipeline, trenchless and tunneling projects. When all things are considered, the project team needs to evaluate is the risk greater than the benefit that could be achieved by taking on this action.

INTRODUCTION

It is exciting to participate in the expanding use of tunneling and trenchless technology as more complex and innovative projects continue to be designed and constructed. Underground projects are being completed to surpass even our own expectations. Boundaries are being pushed, high risk projects are being successfully completed, safety and risk management is top priority and innovation is helping to make previously far reaching concepts possible. Breakthroughs include larger diameters, longer installations, complex alignments and construction in the most difficult geology. When we look at project records or the projects receiving the biggest awards in our industry, they are very different from just a decade ago.

There continues to be a strong belief across the country in the value of underground projects to support our infrastructure needs. There are many new pipeline, trenchless and tunneling projects included in proposed transportation and water programs at the federal and state levels where new funding is being requested across the country. In these programs, more complex projects are being proposed every year. The engineering and construction industry is becoming better at evaluating the project risks and taking pro-active steps to manage these risks before problems occur. This allows owners to undertake projects that we use to fear and now have a high confidence in being successful.

OVERVIEW OF RISK PROCESS AND TYPES OF RISKS

Risks are inherent in all projects. Once the risks are identified, there are three options:

1. **Reject the risk** – All things considered, is the risk greater than the benefit to be obtained? In these situations where the risks outweigh the benefits, alternatives may need to be considered such as different alignments, changes in approach or technology to be applied. It is important to realistically face the risk. Most severe claims do not always come from the most complex projects.

2. **Accept the risk** – All things considered, the benefit is greater than the identified risk. The risks are appropriate for the project benefits to be achieved. The project is within today's standards or operating envelope.

3. **Manage/Mitigate the risk** – Use mitigation techniques, share the risk or allocate the risk to another party. The opportunity to minimize risks is typically in the early stages of the project. This includes the feasibility period and preliminary design. As the project moves into final design and construction, there are fewer opportunities to influence the project and the cost of making changes becomes much more expensive.

The key to success is to evaluate the risks early in the project and to conduct a continuous evaluation of risks over the life of the project.

<u>Types of Risks</u>

Project risks can be divided into the following groups:

Technical Risks. Includes factors such as the nature of the project, personnel and company capabilities and experience, construction industry factors, constraints on time and cost, and attributes of the project owner.

In the underground business, some specific examples of technical risks are:

- Ground movement or settlement
- Pipe or manhole settlement
- Movement of above-ground structures
- Failure of pipe or liner
- Slow production or stoppage of equipment
- Inadequate or inappropriate equipment
- Ground contamination

Financial / Contractual Risks. Includes items such as risk of performance, penalties to meet deadlines, liability for failures or problems, consequential damages, and guarantees or warrantees.

Examples of financial and contractual risks seen on recent underground projects include:

- Penalties for failure to meet owner or court ordered deadlines for completion (e.g., EPA Consent Decrees for Combined Sewer Overflow (CSO) Programs or specified completion dates before penalties occur)
- Subcontractors or subconsultants who are not able to perform
- Lengthy payment terms or unable to suspend work when payment does not occur
- Unlimited liability for failures or problems that occur during construction (no financial or time limitation due to problems or failure of the installed system)
- Unpredictable consequential damages (no financial limitation from third party claims impacted by project construction)
- Extended guarantees or warrantees (warranty period outside of industry practice)

Third Party Risks. Includes disruption to residents and loss of public amenity, disruption to business/commerce, disruption to traffic, increased accidents, disruption to adjacent utilities and environmental impacts.

Third party risks include items such as:

- Environmental impacts due to noise, vibration, dust
- Disruption to adjacent utilities including water, sewage and power
- Disruption to businesses and commerce (e.g., from road closures)
- Increased traffic and accidents
- Disruption to neighborhoods and parks

RISK IDENTIFICATION AND EVALUATION TOOLS

This section presents one method for identifying and evaluating risks that may occur during a tunnel or trenchless project. The methodology presented has evolved based upon its use on a variety of large-scale underground infrastructure projects. Table 1 presents an overview of the process which is then followed by examples to illustrate how the methodology is used.

Table 1: Risk Evaluation Approach and Mitigation Plan

Type of Risk	Risk Description	Potential Impact of Risk	Risk Metrics				Risk Mitigation Plan
			P	I	U	RR	

Type of Risk: The three major types of risks referenced earlier include Technical, Financial/Contractual and Third Party. These major categories can be subdivided into other groups such as Community, Design, Geotechnical, Construction, Funding, Permits, Real Estate, Safety, Unforeseen Conditions, and Utilities.

Risk Description: Define the risk and potential risk events. Also, identify the responsible party. The responsible party includes the Owner, Design Engineer, Construction Contractor or Insurance Company.

Potential Impact of Risk: Define what will be impacted by the specified risk. Typical items include schedule, cost, safety and health.

Risk Metrics:

P – Probability or likelihood that the risk will occur. An example of a scale from 1 to 5 is:

1 = Very low probability of occurrence during project execution. Not expected to occur.

2 = Low probability. May occur once during project.

3 = Moderate probability. Will occur at least once during project.

4 = High probability. Expected to occur more than once during the project

5 = Very high probability. Likely to occur several times during the project.

I – Impact or severity of the risk item if it occurs. The range of costs would need to be determined based upon project size. This is an example for a project with construction value in the range of $250 million or more. Define a scale where:

1 = Low severity or no impact on project outcome. Insignificant cost of $0 to $100,000 for this example and schedule impact of less than one week delay of completion date

2 = Does not impact project outcome. Low impact on cost and schedule. $100,000 to $1 million and less than one month delay.

3 = Minor impact on project outcome. Considerable impact on cost and schedule. $1 million to $5 million (2% of construction cost) and one month to three months delay.

4 = Significant impact on project outcome. Cost impact $5 million to $25 million (less than 10% of construction cost for this example) and three months and six months delay.

5 = Very high severity or major impact on project outcome. Greater than $25 million (greater than 10% of construction cost) and schedule impact greater than six months delay.

U – Urgency of the risk item if it occurs where:

1 = low urgency (immediate action not required)

5 = medium urgency (action required within 24 hours)

10 = very urgent (action must be taken immediately)

RR – Relative Risk = P * I * U (using above scales highest score is 250)

Risk Mitigation Plan: Describe the response to be taken. Common choices are: Avoid the Risk, Reduce the Risk and Transfer the Risk to another party. Describe responsible organization, responsible individual and target completion date.

Tables 2 and 3 show some examples of the Risk Evaluation Approach and Mitigation Plan for a major tunneling project. These tables are only a sampling of the types of risks that could be encountered.

Frequently for an engineering firm, the first step in a risk evaluation is to evaluate its internal risks for undertaking the project. Firms may use this type of risk analysis to make a "go" decision before bidding the project.

Table 2: Examples of Engineering Risks to Make a "Go" Decision Before Submitting a Proposal (Responsible Party: Design Engineer)

Type of Risk	Risk Description	Potential Impact of Risk	Risk Metrics				Risk Mitigation Plan
			P	I	U	RR	
Technical: Design Risk	Project will require 25 employees in the hub office for the design phase of the DB (Design Build) project. This exceeds the available resources in the region.	Significant costs associated with hiring, relocation and/or temporary duty. Schedule impacts if the staff is not delivered in a timely fashion.	3	4	8	96	*Reduce the Risk* Schedule and cost impact. Develop estimates to realistically account for hiring, relocation and travel costs. Implement a project mobilization plans at Notice of Award to clarify time requirements.
Technical: Design Risk	Design performance issues	Potential E&O (Errors & Omission) Claim	2	4	6	48	*Reduce the Risk:* Cost impact. Staff project with personnel who have appropriate experience. Develop and follow QA process. Have outside checker review design.
Technical and Contractual: Requirement for Design Subconsultants	Contract requires 35% use of M/WBE (Minority and Women Business Enterprise) subconsultants to perform work	Loss of control on several design items. Availability of personnel to meet schedule requirements.	4	4	7	112	*Reduce the Risk:* Schedule and Cost Impact Assist subconsultants with develop of detailed work plan and design submittal register. Conduct weekly project reviews with subconsultants.

Using the information in Table 2, executive management of the firm would be able to make a better decision on whether or not to bid this opportunity. Two of the three risks listed in Table 2 are significant. Management would need to be convinced that the mitigation plan was adequate to overcome these issues and still lead to a successful and profitable project.

The information in Table 3 would be developed during the design process and presented to the owner. These risk items may be cited in bid documents to allow the contractor to price these items as part of the bidding process.

Table 3 Example of Risk Evaluation Approach and Mitigation Plan for a Tunnel Project

Type of Risk	Risk Description	Potential Impact of Risk	Risk Metrics				Risk Mitigation Plan
			P	I	U	RR	
Technical: Permitting Risk (primary)	Delay in right of way and property acquisition required for construction of shaft sites Responsible Party: Owner	Delays start of construction	3	3	5	45	*Reduce the Risk and Transfer the Risk* (where possible) Schedule impact. Meet with permitting agency and reevaluate schedule and NTP.
Technical: Construction Risk	TBM (Tunnel Boring Machine) downtime: wear of cutter disks and replacement of equipment Responsible Party: Construction contractor	Excessive equipment downtime resulting in project delay and potential change orders	5	3	7	105	*Reduce the Risk* Schedule and cost impact. Provide safe havens in the schedule for periodic cutter head maintenance. Use high quality cutter disks and materials.
Contractual / Financial: Commercial Risk	Substantial deficiencies in Geotechnical Baseline Report (GBR) Responsible Party: Design Engineer	High claims exposure including change in conditions and schedule impacts	2	5	5	50	*Reduce the Risk* Cost impact Conduct independent third party review of GBR

Based upon the information in Table 3, the TBM downtime is the most serious risk and will need to be addressed as part of the construction approach and contingency planning. There are obviously many additional risks that could occur during this project. This approach should be used to evaluate and then compare all of the likely risks.

The approach shown in Tables 2 and 3 could be adapted for many tunneling and trenchless projects. The risk register is not a static document. The risks should be regularly reevaluated. A good practice is to reevaluate the risks as part of the monthly project review meetings. Frequently risks that have high scores may change (and possibly become lower) as the design progresses and more information is obtained.

RISK MITIGATION TOOLS

As cited in the examples in the earlier section, there are many tools available to help mitigate the identified risks. However, the first step is to carefully identify these risks early in the work program.

During the design, procurement process and construction oversight, engineers can use several methodologies and tools to mitigate risks. These design mitigation tools include:

- Early utility investigations
- Geotechnical monitoring program
- Value engineering session
- Independent peer reviews
- Performance based specifications
- Road and building condition survey prior to construction
- Equipment specification (e.g., horizontal directional drilling equipment, microtunneling or tunnel boring machine)
- Construction monitoring program

The engineer needs to evaluate which of the above mitigation tools are appropriate for the project and determine when they should be applied. There is not a one size fits all approach to the use of these tools.

Here are some of the tools that can be used to allocate risks to the project participants:

- Geotechnical Data Report (GDR) – Document developed by the designer (or geotechnical engineer working for the designer) that presents the factual subsurface information obtained during the exploration and design phases of the project. Typical information includes boring logs and results from field and laboratory tests performed.
- Geotechnical Baseline Report (GBR) – Addresses the design team's interpretation of the subsurface conditions that contractor will likely encounter. Establishes a contractual basis for the allocation of geotechnical conditions.

- Environmental Baseline Reports (May be combined with GBR) – Similar to the GBR the EBR provides an interpretation of the environmental conditions that the contractor will likely encounter.
- Differing Site Conditions (DSC) Clause – This clause is frequently included in the general conditions of the contract. The use of the DSC dates back to the 1920's when it was referred to as changed conditions. The DSC clause provides a method for the contractor to notify the owner and seek relief when conditions are materially different than shown in the contract document.
- Escrow Bid Documents – The bid documents are placed in custody with a third party in case of a dispute during construction of the project. If a dispute occurs, the documents can be used as a basis to determine how an item was bid and be used to seek a fair resolution between the owner and contractor.
- Dispute Review Boards- Typically an individual or group of experts gathered to provide an independent an impartial evaluation of a dispute between an owner and contractor, and render a decision as to which party's position is correct based upon facts.
- Specify Type of Defined Obstructions- Contractual methods for handling unforeseen conditions such as obstructions have been subject to great debate. Three methods for handling obstructions include: 1. Contractor is responsible for handling all obstructions. 2. The owner of the project assumes the risk for handling obstructions. 3. Contractual language is placed in the contract documents requiring a shared risk approach.

There has been controversy with the use of risk allocation tools. It is important that these tools are implemented by experienced individuals who are knowledgeable about their use and implementation.

There are also several items available to help mitigate performance risks. These include:

- Pre-qualify contractors based upon a defined set of criteria
- Disclose all subsurface information to bidders
- Allow sufficient time for bidders to prepare bid packages
- Request methods statements as part of bid packages
- Separate the project into various contract packages and execute separate contracts for early work on the site
- Contractor maintains responsibility for selection of means of construction, equipment and methods for execution of the project

In the right situation, each of the performance risk mitigation tools can be effective. Many of these techniques are common sense approaches but sometimes forgotten or not used due to time and budget pressures.

CONCLUSIONS

A critical element of every subsurface project should be the identification, allocation and assessment of risk. This paper presented an approach for identifying and evaluating risks for underground projects. Once the risks are known, it is important to determine which risks can significantly impact the project, allocate the risks to the most appropriate party and then develop a plan to manage or mitigate the risks. Underground projects are frequently complex and risky. Owners, engineers and contractors need to establish equitable methods for allocating risk in order for projects to be successfully constructed at an appropriate price. As the industry continues to undertake more challenging projects, it is critical to have the best tools and methodologies for managing risks.

Assessing the Condition and Consequence of Failure of Pipes Crossing Major Transportation Corridors

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Abstract
There are uncased aging pipes crossing major interstate freeways and congested railroad corridors throughout the United States. In the case of the Portland water system, more than 100 uncased crossings have been identified. These pipes are what many consider "high consequence" pipes as they potentially pose greater financial impacts than those in the general pipe population, not only to the utilities that manage them, but to the public as well, as the effects of failure may be felt widespread throughout society. To capture the full effects of such an event, Triple Bottom Line (TBL) impacts to society are quantified in the risk management methodology. Understanding the failure modes and root causes of failure, condition, and consequences of failure of pipes crossing major transportation corridors will assist utilities in better managing these high consequence assets. The likelihood of failure can be refined by obtaining knowledge regarding the pipe's operational and environment conditions, and historical performance. Estimating the financial consequences associated with main breaks has been facilitated by the development of transportation disruption models. By refining the likelihood and consequence of failure estimates, Portland Water Bureau (PWB) has been able to establish the business case for mitigation measures, including the potential for continued monitoring, additional valves, or pipe rehabilitation or replacement options.

Introduction
Portland Water Bureau (PWB) supplies drinking water to more than 585,000 customers throughout Portland and an additional 371,000 customers through regional wholesale providers. Portland's drinking water is primarily surface water supplied from the Bull Run watershed. Groundwater from the Columbia South Shore Well Field (CSSWF) is used as an emergency back-up and for supply augmentation during the summer high demand and low precipitation season.

The PWB approach to most pipe assets is to manage multiple failures (leaks, breaks) and wait until it makes economic sense to replace the pipe. High consequence pipe failures, by their definition, are to be avoided or steps taken to mitigate the impacts of a failure. For the vast majority that do not have high consequence of failure, it is far cheaper to repair the pipe multiple times than to replace it. Of the 2,300-miles of pipe in the system, less than 10% are high consequence. These are the general categories PWB is using:

1. Under-crossings of major highways (in town)
2. Under-crossings of major railroad lines (in town)

3. Pipes hanging off of bridges that cross above major highways, railroads and rivers/streams (in town)
4. Primary (or sole) supply lines to most critical services (hospitals, top 20 water users – individual location)
5. Transmission mains that cross under rivers
6. Transmission mains that are hydraulically critical
7. Conduits[1] in locations designated as "High Vulnerability Rating" where a leak or break in the pipe may cause multiple conduits to fail
8. Conduit locations where conduits located within or cross a high traffic roadway, in easements through private property where development has occurred, within environmental zones, or where conduits cross high pressure gas lines.

Inventory of High Consequence Assets

One of the first steps in pipe asset management is to identify which pipes pose a high consequence of failure. What makes a pipe consequential? One way to characterize high consequence is in terms of the dollar impacts of the failure. PWB has classified high consequence pipes as those in which failure may incur a million dollars or more in societal or internal costs. Uncased pipes that cross under major freeways, highways, and railroads, and those suspended from bridges represent some of the highest quantified consequence of failure events as disruptions to major roads or railroads may result in significant triple bottom line impacts. The GIS is commonly used in the selection process.

Table 1 describes the number of uncased pipes crossing below major highways (defined by traffic count); pipes crossing below key mainline railroads including two main north-south lines carrying Amtrak trains, inter-city regional rail service, high priority freight trains, and general freight; and pipes on or suspended from bridges.

Table 1: High Consequence Pipes by Location

Location	Traffic Count (vehicles/day)	Uncased Under Crossings	Uncased Crossings Above
Interstate 84	174,000	8	4
Interstate 5	145,000	29	14
Interstate 205	140,000	9	11
Highway 26	125,000	3	1
Interstate 405	115,000	16	11
Highway 99 E	50,000	11	1
Highway Total		76	42
Rail Lines	N/A	50	22
Total		**126**	**64**

[1] Conduits refer to the three large diameter (44 – 60-inches) primary supply lines that convey water from the Bull Run Watershed into town.

Failure Modes and Root Causes of Failure

The mode of failure is important in assessing the likelihood of failure. Failure modes may be because of physical, capacity, obsolescence, or level-of-service problems. For the purposes of this report, the focus will be on physical failure modes (structural).

Pipe failures, such as leaks and main breaks, are excellent opportunities to capture first-hand information on pipe and also environmental conditions to use in long-term pipe management planning, as these events can be indications of the condition of buried infrastructure. How the pipe fails is important as the type of failure determines the magnitude of the impact. A pinhole leak on a steel pipe creates much less of an impact than a horizontal break of a cast iron pipe. A small leak could become a large leak and could undermine a pipe, but, under most circumstances, a horizontal break will cause much greater damage.

The type of failure also dictates whether there are condition assessment techniques that can identify early evidence of an impending failure. For example, joint separation of a cast iron pipe may be detected as a small leak before a major problem occurs. In contrast, a circumferential failure caused by settlement is not going to be identified with current condition assessment technology. Many condition assessment techniques are able to detect changes in wall thickness, which is important for corrosion-caused failures. The majority of failures in the PWB pipe network are not attributed to corrosion. This limits the opportunity to do condition assessment.

Table 2 lists the failure mode percentages, by pipe material and size, as recorded in the Computerized Maintenance Management System, for the past five years.

Table 2 – Pipe Failure Modes by Cohort, 2010 - 2015

Pipe Material & Size Grouping	Seal failure	Vertical break	Horizontal break	Pinhole leaks	Other	Total
Cast Iron 8-inch or less	2%	74%	6%	5%	13%	491
Cast Iron 10- to 12-inch	21%	24%	17%	7%	31%	29
Cast Iron 14-inch or greater	33%	7%	13%	0%	47%	15
Ductile Iron 8-inch or less	7%	72%	0%	3%	17%	29
Ductile Iron 10- to 12-inch	50%	0%	0%	0%	50%	2
Ductile Iron 14-inch or greater	0%	0%	0%	0%	100%	1
Steel 8-inch or less	0%	29%	6%	56%	9%	133
Steel 10- to 12-inch	0%	100%	0%	0%	0%	1
Steel 14-inch or greater	0%	0%	0%	100%	0%	3

Risk Assessment

In economic risk analysis, the cost of risk is measured in terms of the probability of failure and the cost of the consequences of failure. The framework for assessing business risk exposure is the triple bottom line (TBL) methodology that examines the social, environmental and financial impacts. TBL includes factors such as safety, disruption to the community, adverse effects on the environment, property damage, traffic and other social disruptions, costs to businesses, image of PWB, and other factors in assessing the risk exposure.

The reduction or avoidance of risk must be quantified and have a dollar value attached to it in order to compare the benefit of reducing risk to the cost of providing the higher level of service to customers. To obtain these outputs, the following tasks for determining inputs are required:

- Identify the risks
- Determine the likelihood (probability) of failure as the asset ages
- Quantify the consequences of failure in financial terms
- Determine the costs (resources) required to mitigate the risk

Comparing the benefits of reducing risk cost to the actual cost of the project is one of the main tools used in a business case analysis. The key is to determine when replacement can be justified economically as well as the level of maintenance that will cost effectively reduce the risk of failure.

Likelihood of Failure

The likelihood of failure (LOF) of a pipe is based on a Weibull estimation for a group of similar pipes, or cohort. The median time to failure (MTF) of the cohort's Weibull curve is defined as its "expected useful life." Weibull curves are probability distribution functions of the original population's failure rate. Failure rate of the remaining population[2] is calculated from the Weibull curve. Fitting historical failure[3] data for each cohort to the remaining population failure curves is the basis for estimating the useful life of a pipe cohort. The failure curve of the remaining population is then used to determine the probability of failure of the pipe at a corresponding age. Figure 2 represents a mean time to failure Weibull curve and the corresponding failure rate of the remaining population.

[2] It was assumed that a pipe that had failed was taken out of the "remaining' population although it still may be in use and has not yet removed.
[3] "Failure" is defined as two leaks or a single break on a continuous 300-feet section.

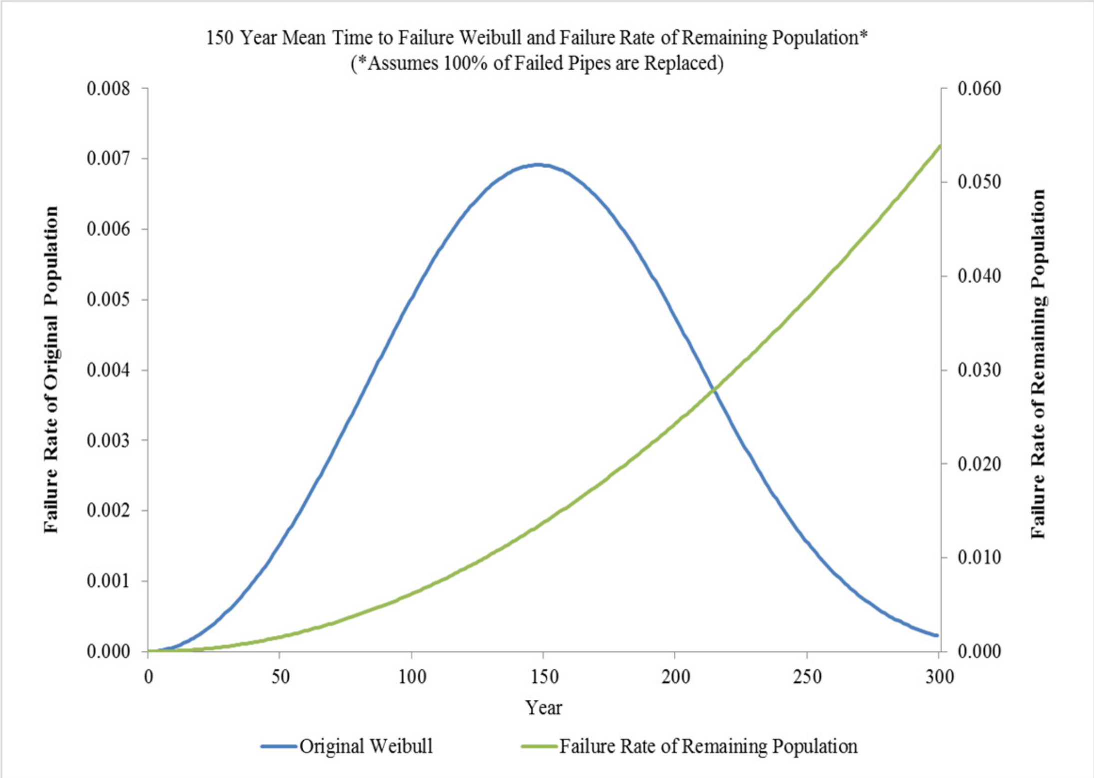

Figure 2 – Weibull Curve and Failure Rate of Remaining Population Curve

The useful life estimates were developed for the Distribution Mains Asset Management Plan, displayed in Table 3.

Table 3 – Weibull Curve Useful Life Estimates for Select Main Materials

Pipe Material	Construction Date	Size (inches)	Useful Life Estimate[1]
Cast Iron	Pre-1930	≤ 2	110
Cast Iron	Pre-1930	4 – 12	210
Cast Iron	Pre-1930	≥ 14	250
Cast Iron	1930 – 1954	≤ 2	110
Cast Iron	1930 – 1954	4 - 8	190
Cast Iron	1930 – 1954	≥ 10	210
Cast Iron	After 1954	≤ 2	35
Cast Iron	After 1954	4 – 8	110
Cast Iron	After 1954	≥ 10	210
Steel[2, 3]	< 1930	≤ 2	85
Steel	< 1930	4 – 12	150
Steel	< 1930	≥ 14	200
Steel	1930 - 1969	≤ 2	65

Steel	1930 – 1969	4 – 12	120
Steel	1930 - 1969	≥ 14	180
Steel	After 1969	≤ 2	50
Steel	After 1969	4 – 12	150
Steel	After 1969	≥ 14	200
Ductile Iron[3]	Pre-1966	≤ 8	140
Ductile Iron	1966 to present	≤ 8	260
Ductile Iron	1966 to present	≥ 10	300

1. Useful life estimate is the median time to failure on a Weibull distribution (i.e. 50% of the population is expected to "fail."
2. Steel includes galvanized steel.
3. If CP was included at install add 100 years to useful life. If CP was added later, add 50 – 100 years on to useful life, depending how long after pipe installation the CP was added.

Condition Assessment Results

Since risk is both the consequence and the likelihood of the failure, it is important to assess the condition of the high consequence asset to determine the extent of risk. The Weibull curves give an average or general estimate of a pipe's useful life; the assessment of pipe segments in terms of deterioration, or evidence of impending failure, will give a segment a specific estimate of remaining life.

Condition assessments and planning studies are an integral part of the approach to managing risk and developing the bureau's Capital Improvement Plan (CIP). The bureau has conducted condition assessments of high pressure pump mains, pipes on bridges, pipes under freeway and railroad crossings, in key operational areas, and during pipe failure events.

A total of 71 pipes (approximately 6 miles) suspended from bridges were visually inspected between March and October of 2014. Pipe risk ratings were updated following the inspections and information related to the condition assessment findings was used to recommend follow-up inspection, maintenance, repairs, or further analysis. One pipe was found to be in danger of imminent failure. The pipe was repaired and the risk was reduced to an acceptable level until a permanent solution could be implemented. A previous failure at this location caused millions of dollars in societal and internal costs.

Over 30 miles of high consequence pipes have been evaluated for leaks using innovative technologies, such as acoustic leak detection, broadband electromagnetic, and Pure Technologies Sahara®. Finding leaks has enabled the bureau to adjust the pipe's expected useful life and make more informed mitigation decisions. Table 4 lists the results of the acoustic leak detection assessments completed over the past five years.

Table 4 – Results of Acoustic Leak Detection (2009, 2010, 2013)

Pipe size and material	Locations	Miles surveyed for leaks	Leaks detected	Leaks per mile
Steel, > 50-in.	Conduits	21 miles	1	.05
Steel, 16-48-in.	Mostly transmission mains	4.5 miles	0	0
Bar wrapped concrete cylinder pipe, 66-in.	WCSL, also I-205 crossing for conduit	2.7 miles	5	1.9
Cast iron, 8-36-in.	Some transmission mains, mostly high consequence crossings	3 miles	1	0.33

Pipe failures, or breaks, are frequently a key data input considered by utilities in deciding which pipe to replace. The occurrence of a break starts the process of gathering useful data, given that when the break is repaired, an excavation is generally made, exposing the pipe so that workers can repair the pipe. This repair event is, of course, an excellent opportunity to gather additional data on the pipe condition in that area at relatively minimal cost (Water Research Foundation, 2015).

Consequence of Failure

The consequence of an asset failure relates to whether PWB can meet a particular level of service or whether the failure has an environmental or social impact that is external to PWB operations. There are seven broad consequence categories and 27 more specific impact categories identified and used in the process, each with a series of adverse impacts which would be caused by a failure of the asset. The seven consequence categories include regulations, impacts to supply, public confidence, social impacts, environmental impacts, loss of revenue, and large expenditures.

Estimating the Consequence of Failure

Traffic Impact Valuations

Estimating the consequence of failure may require research and evaluation. PWB analytics suggests that the impacts resulting from pipe failures beneath major highways may result in significant costs to society. Consequences include the cost to repair the damaged roadway, which for highways and freeways is estimated according to the type of failure (horizontal/vertical breaks, leaks), and could be in the hundreds of thousands of dollars for major roads. In addition, it is assumed that as soon as roadway repairs are completed, a new encased main would be tunneled under the roadway, with costs likely in the millions of dollars, depending on the circumstances. Typically, reduced fire flow, low pressure, and water outages have minimal quantifiable impacts given the amount of redundancy in the distribution system and relatively small number of impacted water supply customers (compared to the large number of citizens using the road).

Triple-bottom-line (TBL) consequences of failure include the social costs of traffic delays. It is assumed horizontal and vertical breaks require a minimum of three to five days to repair and repave the road, and for these large breaks the damage to a highway or freeway, and the subgrade beneath the roadway, is assumed to be large enough to require all lanes in one direction to be closed. Traffic delays are assumed per vehicle depending on the location of the failure, the detour routes available, and the average daily volumes of vehicles using the roadway during the analysis period. PWB Asset Management guideline values of $33.50 for the travel delay per vehicle per hour on intercity highways and $22.50 per vehicle per hour on local roads are based on U.S. Department of Transportation guidelines. **Impact** = $33.50 x Delay Duration per vehicle (hrs) x # Vehicles for Entire Delay Period. Table 5 describes the process used to evaluate the consequence of failure of an 8-inch main crossing Interstate 5 (I-5).

Table 5 – 8-inch Steel Pump Main Crossing under I-5 Consequence of Failure Example

Type of failure	Traffic Delay Costs	Pavement Repair Costs	Micro-Tunnel New Main Under 1-5	Total TBL Costs	CMMS Failure Mode % Probability	Weighted Consequence Costs
Horizontal Break	$14,447,000	$1,500,000	$1,200,000	$17,147,000	6%	$1,028,820
Vertical Break	$11,674,000	$1,000,000	$1,200,000	$13,874,000	29%	$4,023,460
Leak	$8,901,000	$500,000	$1,200,000	$10,601,000	65%	$6,890,650
					Total	**$11,900,000**

Railroad Impact Valuations

Impacts resulting from pipe failures beneath major railroad lines may incur significant TBL consequences as well. The bureau's Asset Management group developed a methodology to estimate the economic impacts of disruptions to railroad services, such as freight and passenger transportation, resulting from water main failures.

The economic impact of a disruption depends primarily on the commodity characteristics, the characteristics of the disruption, and the costs associated primarily with transport and logistics and the type of inventory (National Cooperative Highway Research Program[4] [NCHRP]).

Commodity characteristics under consideration include the type and tonnage of commodities, the value, or value class, and the commodity origins and destinations. Commodity data were obtained from the 2010 Oregon Rail Study by Oregon Department of Transportation (ODOT).

For the purposes of this analysis, a disruption may be either slowing or halting of rail traffic. The key assumptions are that a disruption would be brief, (~24- hours), in addition to the duration of the water main and railroad repair; it would be small on the geographic scale, limited specifically to Oregon's freight economy; and freight would not be diverted to another route or transportation mode. The following assessment was made to quantify a disruption to the railroad network in a specifically congested section within the Portland Corridor where no alternate routes exist.

[4] Report 732, Methodologies to Estimate the Economic Impacts of Disruptions to the Goods Movement System, 2012.

Economic impacts to passenger rail travel are considered in the methodology. A review of ODOT and Washington State Department of Transportation (WSDOT) passenger rail reports revealed that the impacts from a single train being delayed in the Portland corridor would be experienced within the entire network from southern Oregon to Vancouver, British Columbia. Table 6 shows Amtrak annual revenue and an estimation of daily service interruption economic impacts resulting from a disruption in the Portland corridor.

Table 6 – Amtrak Service Interruption Economic Impact

11 Trains on Amtrak Cascades Operate Daily Between Eugene and Vancouver, BC	Annual Ridership	Ticket Revenue	Daily Revenue Loss[1]	Passenger Time Value of Delay[1,2]	Daily Service Interruption Economic Impact[2]
Totals	734,200	$26,620,000	$72,900	$28,100	$101,000

1. Assumes one hour delay, $14 per passenger per hour
2. Assumes full refund or full trip revenue loss.
Source: Amtrak Cascades 2013 Performance Data Report, WSDOT

Default values for estimating economic impacts to freight are shown in Table 7 below. Inventory costs are cost increases for commodities delayed. Typically, such costs are measured against the market value of freight as a function of the delay, measured in freight ton-hours (NCHRP, 2012). Disruption costs can be estimated by multiplying the volume of freight by the dollar value of freight delayed (the Inventory Cost) by the duration of the delay.

Table 7 – Portland Corridor Disruption Economic Impact

Value Class	% of All Oregon Freight	Tons of Value Class per Corridor per Day[1]	Inventory Cost per Ton-Hour[2] (Class 1 RR)[3]	Daily Disruption Cost
1. High-value manufacturing	12%	18,572	$0.22	$98,000
2. Low-to moderate-value manufacturing	37%	57,263	$0.19	$261,000
3. Low-value bulk commodities	28%	43,334	$0.14	$146,000
4. Perishable agriculture	23%	35,596	$0.23	$196,000
			Freight Total	$701,000
			Amtrak Total	$101,000
		Total Daily Disruption Cost		**$802,000**

1. Based on 56.5 million tons annually through the Portland Corridor (ODOT Rail Study, 2010).
2. Does not take into account the supply chain response to external forces (see discussion below).
3. The Surface Transportation Board classifies freight railroads by their gross operating revenues. Class I railroads are those with annual gross revenues exceeding $401.4 million. Two Class I railroads operate in Oregon, UPRR and BNSF, which handle approximately 90% of the region's freight.

The disruption costs used in the methodology are limited to the tonnage through the Portland corridor, however, a disruption in the Portland area would likely be experienced by most railroads throughout the entire Oregon region. The Portland corridor is densest in terms of rail activity and serves as the hub of most rail operations in the state.

Not included in the disruption costs are PWB internal & repair costs, economic impacts due to disruptions to vehicular traffic, environmental impacts, and supply chain disruptions to short line railroads, the Port of Portland, and the Portland International Airport; manufacturing and economic production interruptions; losses from reduced sales, employment, wages, and gross domestic product (GDP); and litigation costs (external and internal). Thus, the Inventory Costs in Table 7 used to estimate freight disruption costs should be considered low.

Table 8 displays estimates of the consequence of failure costs associated with the failure of a 10-inch cast iron main crossing the congested Portland railroad corridor.

Table 8 – 10-inch Cast Iron Main Crossing under Railroad Consequence of Failure

Type of Failure	Railroad Disruption Cost / Day	Days of Delay	PWB Repair Costs	Consequence Costs	CMMS Failure Mode % Probability	Weighted Consequence Costs
Horizontal Break	$802,000	2	$100,000	$1,704,000	21%	$358,000
Vertical Break	$802,000	1	$25,000	$827,000	24%	$198,000
Leak	$802,000	0.5	$10,000	$411,000	28%	$115,000
					Total	**$670,000**

The consequence cost is then multiplied by the Weibull failure rate of the remaining population to determine an annual risk cost. The annual risk costs are then summed over the duration of the remaining expected useful life of the pipe to determine the present value of the risk cost. This information is presented in Table 9. The utility can then compare the risk cost to the project cost to justify spending to mitigate the risk.

Table 9 – Present Value of Risk Cost of 10-inch CI Main Crossing under Railroad

Year (age of pipe)	Weibull Failure Rate of Remaining Population	Consequence of Failure	Risk Cost	Present Value of Risk Cost (3% discount)
109	0.046315	$670,000	$31,031	$30,127
110	0.047172	$670,000	$31,605	$29,791
111	0.048038	$670,000	$32,185	$29,454
112	0.048911	$670,000	$32,771	$29,116
113	0.049793	$670,000	$33,361	$28,778
114	0.050682	$670,000	$33,957	$28,439
115	0.051580	$670,000	$34,559	$28,099
Fast forward to year 200 of the pipe's existance…				
200	0.157581	$670,000	$105,579	$6,959
201	0.159184	$670,000	$106,654	$6,825
202	0.160797	$670,000	$107,734	$6,693
203	0.162417	$670,000	$108,820	$6,564
204	0.164046	$670,000	$109,911	$6,437
205	0.165684	$670,000	$111,008	$6,312
206	0.167330	$670,000	$112,111	$6,189
207	0.168985	$670,000	$113,220	$6,068
208	0.170648	$670,000	$114,334	$5,949
209	0.172320	$670,000	$115,454	$5,832
210	0.174000	$670,000	$116,580	$5,718
			Risk Cost Reduction	$1,610,000

Conclusions

A firm understanding of the failure modes and root causes of failure, condition, and consequences of failure of pipes crossing major transportation corridors will assist utilities in better managing high consequence pipe assets.

Collecting applicable pipe failure data will eventually reveal information about the performance of a utility's piping system, potentially leading to the development of estimations of pipe cohorts' useful lives. An appropriate condition assessment technology can be used to refine the remaining useful life estimation of a specific pipe, and to make more knowledgeable decisions regarding the timing of mitigation actions.

Using transportation disruption models developed from federal and state transportation economic and commodity data has enabled PWB to better quantify the financial impacts of a main break beneath a major highway or railroad. Although intangible costs are very difficult to quantify, transportation disruption costs can be estimated using data available, specifically the economic costs of delays using average freight and passenger travel time values. The methodology described herein is considered a strong basis for the business case for mediation measures. Recommended mediation actions include:

- Exercise valves to full closure. Many valves are turned to 50% of full closure, which means the vault is accessible, the valve is accessible (sediment does not fully fill the vault), and the valve turns. It does not mean the pipe will seal when the valve is closed.
- Add valves on the pipe. There are situations where multiple valves need to be closed, or the distance from the uncased crossing to the nearest valve is considered too great and there are relatively convenient locations to add a valve. An alternative approach is to add a second valve beyond the first valve for closure.
- Conduct condition assessment. Some failure modes lend to early warning through condition assessment. The bureau has contracts to perform remote and intrusive leak detection, intrusive pipe wall thickness and internal pipe video recording.
- Replace pipe. There are situations where the existing uncased crossing is high risk and warrants risk mitigation involving either replacing the pipe with a cased crossing, or adding internal lining (where the existing pipe becomes the "outer casing."

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Risk Model for Large-Diameter Transmission Pipeline Replacement Program

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Abstract

The East Bay Municipal Utility District (EBMUD) is a major metropolitan water district with approximately 335 miles of large diameter pipelines (LDP) defined as any pipeline 20 inches and greater in diameter. EBMUD's past approach for selecting LDP replacement candidates has been based solely on analysis of leak history data. In order to shift from this reactive approach to a more proactive replacement program, EBMUD recently completed the Large Diameter Pipeline Master Plan which prioritizes replacement pipelines based upon a risk model. The risk model ranks each LDP segment based on its risk score which is a product of the individual pipe segments likelihood and consequence of failure scores. The likelihood of failure criteria includes the LDP condition (age, material, joint type, lining, coating, and leak history) and hazards the pipeline is exposed to (seismic activity, liquefaction, landslide, floods, tsunami, and sea rise). The consequence of failure criteria includes collateral damage concerns, access issues, customer impacts, and system hydraulic importance. The risk model was calibrated and verified based on input from Operations and Maintenance (O&M) staff. Hydraulic modeling and planning criteria were used to cluster individual pipe segments into practical replacement projects. The risk model will be updated on an annual basis. The updated results will be compared against the previous year's results to determine if a change in replacement priority is recommended. The risk model is a more comprehensive, proactive method of selecting LDP replacement projects for the capital improvement program.

INTRODUCTION

The East Bay Municipal Utility District (EBMUD)'s 330 square mile service area is located in eastern San Francisco Bay Area serving approximately 1.3 million customers. The EBMUD system includes over 4,000 miles of treated water distribution pipelines and approximately 335 miles of large diameter pipelines (LDP). EBMUD defines LDP as any pipeline 20 inches and greater in diameter. These pipelines are critical transmission mains that are difficult to remove from service, which makes inspection, repairs, and replacement challenging and expensive.

The previous method of LDP replacement focused on an analysis of leak history data and cost/benefit ratios (the cost of repair versus the cost of replacement). This method is a reactive replacement plan that considers only leak data and does not address the unique characteristics of an LDP, such as pipe age, specific site hazards, hydraulic criticality, and ease of repair. LDPs extend for long distances and replacement costs are high, which skews the cost/benefit ratios and does not prioritize LDP projects adequately. As a result, EBMUD created the LDP Master Plan and replacement program that focuses on the overall pipeline risk.

RISK MODEL APPROACH

The risk model approach calculates a risk score for each LDP segment. The total risk score is the product of the LDP's likelihood of failure (LOF) and consequence of failure (COF) scores per equation below:

Risk Score = LOF Score x COF Score

The LOF score indicates the condition of the pipeline. A high LOF score means the LDP has a greater probability of failure. The COF score indicates the impacts (hydraulically, customer level of service, monetary, etc…) caused by a LDP failure. A high COF score indicates that when the LDP fails, the impacts are relatively more severe. There are several criteria that contribute to the LDP's likelihood and consequence of failure scores. Each has a different weighting factor based on their importance and contribution. The criteria with small weighting factors do not have much influence on the overall risk score, however, they do serve as a "tie-breaker" and provide a basis for ranking pipe segments with similar risk profiles.

Likelihood of Failure. This score attempts to identify LDPs that are currently leaking/failing and have the greatest probability of failing in the near future. There are several factors that contribute to the failure of a LDP. Below is the description of the twelve criteria used to determine the LDP's LOF score. Each of the criteria gets a numerical score between 1 and 10.

Age: This criterion scores each LDP based on the year it was installed. There is a strong correlation between age of pipe and pipe failure. The older the pipeline is the more likely it is to fail and a higher score is given to older LDPs.
Material and Joint Type: EBMUD uses several pipe materials with various joint types. LDPs with unrestrained joints and pipe materials with a higher leak history, such as cast iron, receive a higher score.
Lining: EBMUD uses the following three lining options for LDPs: insulating material (such as epoxy, asphalt, and coal tar), cement mortar, or unlined. EBMUD has observed that unlined pipes and insulated lined steel pipes fail at greater rates and these linings receive a higher score.
Coating: EBMUD uses the following six types of coating for LDPs: uncoated, mortar, tape-wrapped, insulated, mortar with an insulating top coat, insulated with a

mortar top coat. Uncoated pipes and insulated coated steel pipes fail at greater rates and receive a higher score.

Leak History: Leak history is an excellent indicator of the condition of the pipeline. If the pipeline has multiple leaks then the useful life of the pipeline is coming to an end. More leaks result in a higher score. Currently there is not distinction between the various types of leaks.

Seismic Fault Zone: The Hayward and Calaveras fault runs through the EBMUD service area and presents a serious failure potential. The Alquist-Priolo earthquake fault zone maps were used to determine if a pipeline was near a fault zone. Pipeline within the fault zones receive a higher score.

Seismic Fault Crossing: This criterion identifies pipelines that directly cross the fault line and are vulnerable to fault creep and offset. These LDPs receive a higher score.

Seismic Liquefaction: Liquefaction events can cause differential settlement or lateral spreading of the ground along a pipeline resulting in a failure. USGS data was obtained to determine which LDPs were in highly liquefiable soil.

Landslide: Landslides pose a significant threat to buried pipelines due to slope instability. Data from the California Geological Survey was obtained to help determine which LDPs are exposed to landslide risk. This criterion includes landslides that are due to seismic movement, rainfall induced, and slope failure.

FEMA Flood Zone: During a flood event pipelines can be damaged as a result of erosion of the pipeline backfill and excess flood debris can increase surface loads on buried pipelines. In 2009 FEMA produced a Digital Flood Insurance Rate Maps. This map was overlaid on the pipeline network to determine which LDPs are within the 100 and 500 year storm event floodplains.

Tsunami: A tsunami can cause tremendous damage to above ground structures and can deposit tons of debris in the inundation zone. The remaining debris poses a threat to the LDP system by increasing the external loading on the buried pipelines. In 2009 the California Emergency Management Agency created inundation maps based on several "worst case" scenarios for the California coast line. If a LDP is in the inundation area it receives a larger score.

Sea Rise: Sea level rise is a hazard that happens over a long period of time and is a threat to the existing LDPs. Data from the USGS was acquired to help evaluate the consequence of an increase in water surface elevation near the infrastructure.

The total LOF score is the sum of each criterion score multiplied by its corresponding weighting and then totaled, as shown in the equation below:

LOF Score = 15%(*Age*) + 10%(*Material*) + 7%(*Lining*) + 7%(*Coating*) + 25%(*Leak History*) + 13%(*Seismic Fault Zone*) + 7%(*Seismic Fault Creep*) + 5%(*Seismic Liquefaction*) + 5%(*Landslide*) + 2%(*FEMA Flood Zone*) + 2%(T*sunami*) + 2%(*Sea Rise*)

The weighting factors emphasize the criteria that have the greatest impact on the LDP failure. The age, leak history, and seismic fault criteria have the largest weighting factors because they are the largest drivers in the LDP eventual failure.

Consequence of Failure. This analysis identifies LDPs that, in the event of failure, have the potential to cause significant impacts (hydraulically, monetary, reparability, and customer level of service) during an LDP failure. Below is a description of the twelve COF criteria that are used to determine the LDPs total consequence of failure score. Each of the criteria gets a numerical score between 1 and 10.

Proximity to Waterways: GIS buffer tools identify LDPs within 100 feet of a waterway (river, creeks, and streams). A pipeline break near a waterway can have significant environmental consequences and can go undetected since water is usually present in waterways.
Proximity to Railroads: GIS buffer tools identify the LDPs within 100 feet of a railroad. A pipeline failure near a railroad track can disrupt commerce and travel. In addition gas and telecom transmission mains tend to be installed in railroad right-of-ways parallel to railroad track increasing the consequences of a LDP failure near a railroad.
Proximity to Highways: Highways are major transportation routes for shipping goods and public travel. A LDP failure near a highway could risk lives and cause a significant disruption in travel.
Proximity to Major Roads: Major roads are a safety hazard for maintenance crews repairing pipeline breaks due to traffic hazards. Also, pipeline breaks in major roads can significantly disrupt the flow of traffic in a city for an extended period of time.
Proximity to High Priority Facilities: High priority facilities are defined as hospitals, health service facilities, schools, air transportation, communication, and power generation facilities. A pipeline failure near one of these facilities could potentially cause expensive collateral damage by impacting operations to these important facilities.
Inside of Steep Slope: LDPs installed in a steep slope are difficult and costly to repair due to the difficult terrain. Repair crews have difficulty accessing the pipes and transporting equipment. This criterion identifies pipelines within a steep slope and gives them a higher score.
Access Issues: Pipelines within a private right-of-way can be difficult to access which significantly increase the cost of repair. In some instances pipelines travel through properties with structures nearby significantly increasing the risk of large claims during a failure and difficulty to repair.
Pipeline Diameter: This criterion scores the LDP based on diameter. The larger pipelines receive a higher score because they carry more water and are typically hydraulically more important to the system.
Proximity to Distribution Reservoirs or Pumping Plants: LDPs that are near distribution reservoir or pumping plants are critical for delivery of water to the distribution system. This criterion identifies the LDPs that are within 100 feet of a distribution reservoir or pumping plant. This buffer was used to capture the critical inflow and outflow pipelines connecting these facilities to the system.
Backbone Pipeline: Backbone pipelines are pipelines necessary for maintaining storage in a pressure zone. These are pipelines that transfer water between the source (pumping plant or treatment plant) and the reservoir storage or regulator in each

pressure zone. A large percentage LDPs are classified as backbone pipeline, but not all due to large size of some of the EBMUD pressure zones.

Pipeline Consumption: LDPs are transmission mains that travel long distances and have longer distances between valves. As a result large segments of LDPs are required to be out of service during an outage. Customers served directly from a LDP will not have water service for an extended period to repair an LDP break. This criterion identifies LDPs that have service connections.

Repair Record: This criterion scores a pipeline based on its repair record. When a LDP has multiple leaks there are higher costing consequences associated with its more frequent repairs than other LDPs. Also, having repair crews in the same area time and time again produces a negative public perception.

The total COF score is the sum of each criterion score multiplied by its corresponding weighting and then totaled, as shown in the equation below:

> **COF Score** = 10%(*Proximity to Waterways*) + 5%(*Proximity to Railroads*) + 5%(*Proximity to Highways*) + 5%(*Proximity to Major Roads*) + 10%(*Proximity to High Priority Facilities*) + 5%(*Inside of Steep Slope*) + 10%(*Access Issues*) + 10%(*Pipeline Diameter*) + 10%(*Proximity to Distribution Reservoirs or Pumping Plants*) + 5%(*Backbone Pipeline*) + 10%(*Pipeline Consumption*) + 15%(*Repair Record*)

The weighting factors emphasize the criteria that have the greatest or largest consequences during a LDP failure. Six COF criteria have a weighting factor of 10% or greater. These criteria are known to increase the difficulty in repair, have the greatest impact on the customers, or cause claims after a failure.

RISK MODEL RESULTS

A risk score is calculated for each segment of the LDPs. Normalized risk scores for all the LDPs are plotted in a cumulative distribution function below in Figure 1.

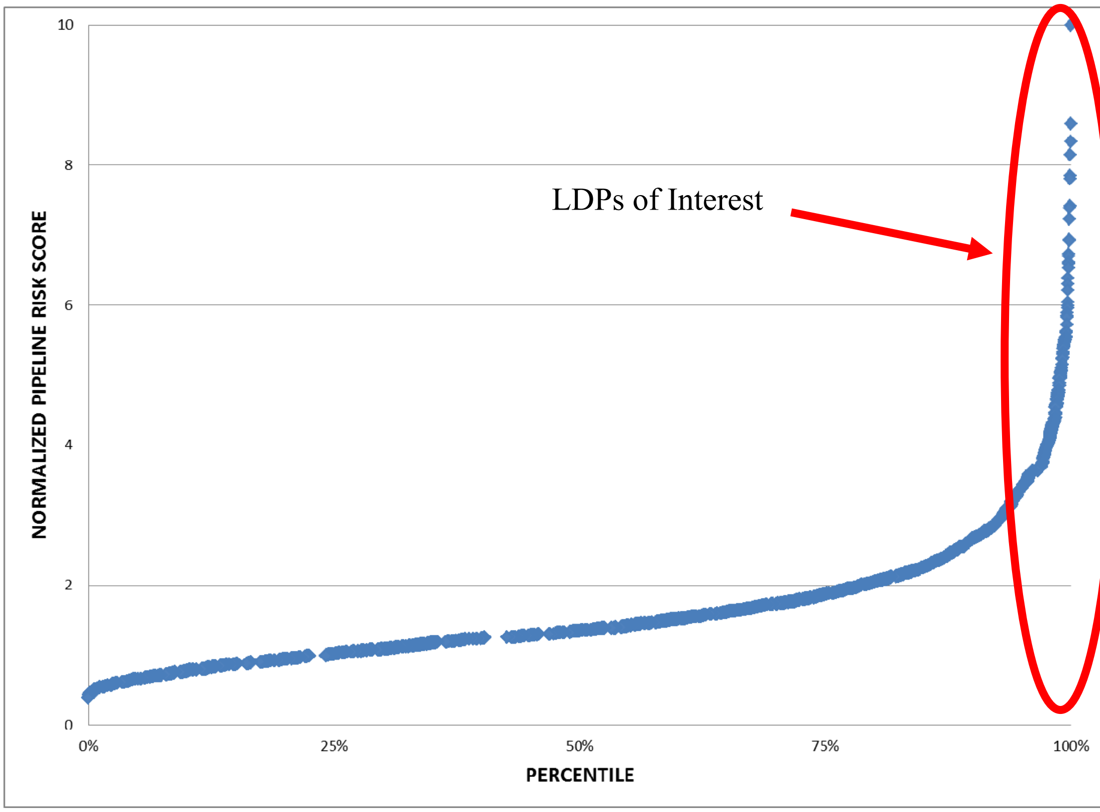

Figure 1. Cumulative distribution function of the normalized risk scores for all LDPs.

Figure 1 shows that the risk scores steadily increases until the 90th percentile. The 90th percentile LDP demonstrate a significant increase in risk scores which corresponds to approximately 30 miles of pipeline. The model has identified the pipes which have a significantly higher risk of failure and these are the LDPs that will be the focus of the replacement program.

The LDP program goal is to replace approximately 3 miles of LDPs per year. This model ranks the pipelines according to risk score and projects will be developed around these top candidates. The risk ranking is the basis for prioritizing LDP projects for the EBMUD's capital improvement program.

Sensitivity Analysis. A sensitivity analysis was completed to determine which likelihood and consequence of failure criteria have the greatest impact on the total risk score and the relative ranking. Simulations were completed where each criterion was varied from -100% (eliminated) to +100% (doubled). The pipelines identified within the 97th, 90th, and 60th percentile of the basecase were compared to the corresponding percentile of the sensitivity simulations. The 97th percentile corresponds to 9 miles of LDPs with the highest risk score, while the 90th percentile corresponds to 33 miles and the 60th percentile corresponds to 158 miles. The largest changes in pipeline risk rankings were observed when comparing the 97th percentile. However, on average these changes only produced a 10% difference in the pipeline ranking. When examining the comparison of the 60th percentile on average there was only 2% to 5% change in pipeline ranking.

Figure 2 is a radar chart of risk model sensitivity analysis results when the likelihood of failure categories were eliminated. It plots a percent in common between the 97th, 90th, and 60th percentile pipelines in the basecase and the -100% sensitivity simulation for all the categories.

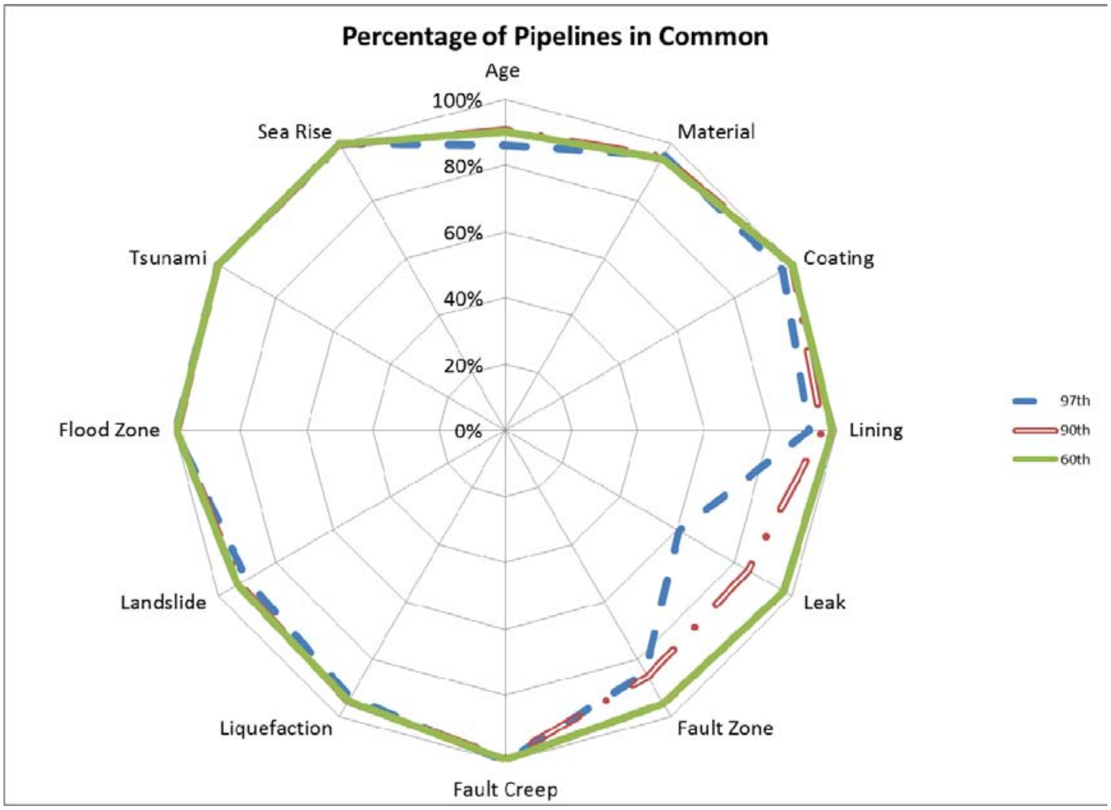

Figure 2. Risk model comparison between basecase and -100% of each LOF criteria

Figure 2 shows that the risk score is sensitive to the subtraction of the leak, age, and fault zone scores, especially when comparing the 97th and 90th percentile. The comparison of the 60th percentile only yields nominal changes in the priority ranking. Similarly, the risk model is the most sensitive to increases in age, leak and fault zone scores. When the LOF scores are increased, the fault zone and age categories produced the largest change in priority, not the leak criterion, because LDPs with the highest risk score typically has a high leak score.

The same process was completed on the consequence of failure criteria. The risk model was most sensitive to the repair record, consumption, waterway, diameter, proximity to pumping plant/reservoir, and right-of-way criteria. Figure 3 is a radar plot of the consequence of failure sensitivity analysis results when the various categories are increased by 100% (doubled).

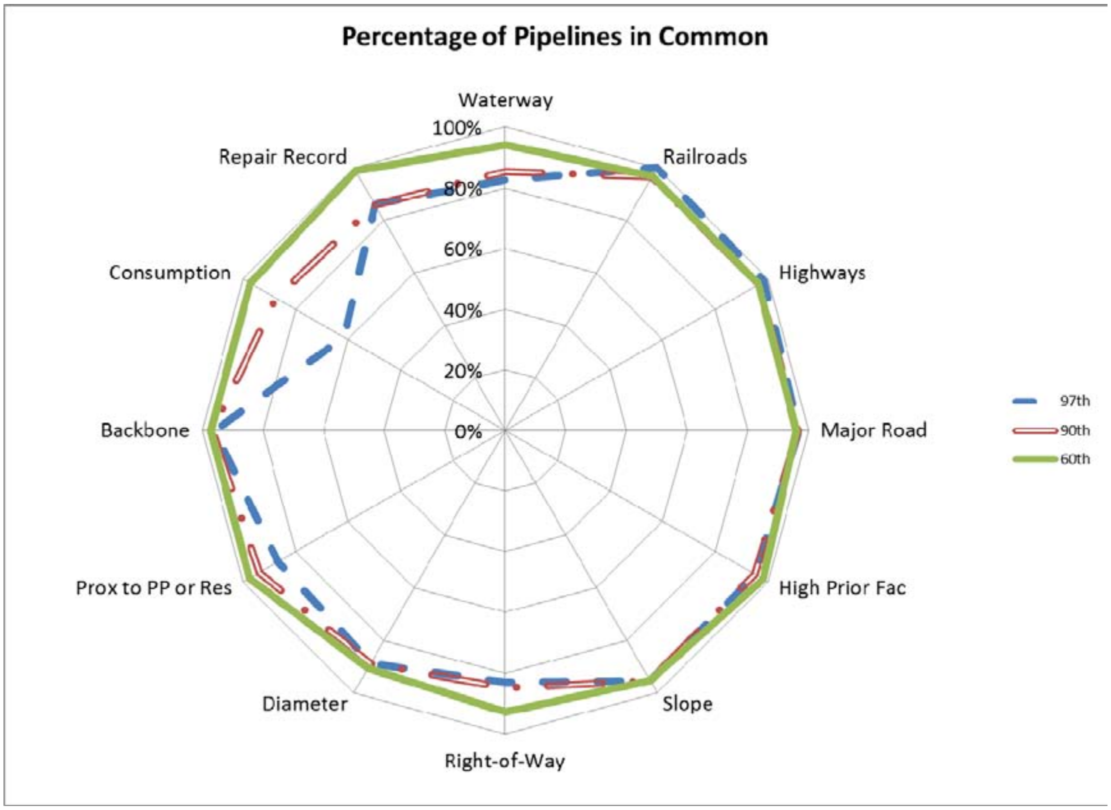

Figure 3. Risk model comparison between basecase and +100% of each COF criteria

Validation. To validate this risk model the results were compared to a recent LDP projects list that were selected based on leak history solely. Of the twelve projects identified in the previous large diameter replacement capital budget nine of these projects were top replacement candidates identified by the new risk model. The reason the other three projects were not included in the top twelve is due to their moderate consequence of failure scores. All three projects were identified due to their high LOF scores, however, the previous LDP project selection methodology did not incorporate consequence of failure into the replacement decision. Now these three projects can be postponed freeing budget to replace higher risk ranked projects.

These results were also discussed with the O&M Division. They provided helpful insight and concurred with the updated replacement candidate ranking.

Limitations. This risk based approach provides a replacement strategy that is justifiable and proactive. It also allows for flexibility to make adjustments to scoring systems and weightings to adapt to potential changes in the system or environment. However, there are some limitations within this risk model, in that this risk model acts as a laser pointer. It identifies a specific segment of a LDP, but does not identify the project boundaries. It is the responsibility of the project manager to use their judgment to determine the extent of the project and which adjacent pipelines need to be replaced to create a cost efficient project. Most of the time, LDPs with a high risk score are clustered together because they have been installed at the same time, using the same material, and exposed to the same environmental conditions. However, there are a few locations where the model identifies isolated LDPs with a high risk score that are connected to low risk pipes making it difficult to create a cost-effective project.

Since this model incorporates both the likelihood and consequence of failure criteria, if a LDP has a very high LOF score, but has a moderate to low COF score it will not be a top replacement candidate. There are a few instances where LDPs have numerous leaks, but are in areas with minimal collateral damage potential. These pipelines are being monitored on an annual basis to check their replacement priority.

FUTURE MODEL IMPROVEMENTS

The LDP risk model is a significant improvement from the previous replacement strategy. However, as data sets improve and the risk model is further integrated with other EBMUD systems, future improvements should be made. Below is a list of some of the proposed future improvements that will improve the risk based model:

1 - Incorporate Hydraulic Modeling Data. Currently, this model uses surrogates for determining the LDP's hydraulic importance instead of hydraulic model results. An all-pipe model for the EBMUD LDP system across pressure zones is currently under development. This model will provide improved hydraulic data that identify pipes that are critical for maintaining transmission through a pressure zone and will be able to quantify the impacts of a pipeline shutdown, such as the level of service to customers. The location of existing valves could be incorporated to quantify the impacts of the failure of specific segments of the LDP.

2 - Incorporate Claims Data. LDP failures can cause a large amount of damage and disruption to the surrounding areas. Some main breaks have resulted in significant damage claims against EBMUD. To improve the consequence of failure aspect of this model incorporating claims data would be a big improvement. Currently, the claims data are not in a usable format but it could become a future enhancement to include LDP history with claims.

3 - Improve Leak Data. The leak data that is used in the model only totals the number of leaks on a pipeline segment. No data is given about the severity or type of failure that created the leak, such as if it was related to a service connection or full-circumferential breaks, etc. This useful information could be incorporated in future versions of the model to improve the likelihood of failure score. Also incorporating how long each leak repair took could be incorporated. This would further refine the pipeline leak score.

CONCLUSION AND NEXT STEPS

The new risk based approach to LDP replacement is a more holistic, balanced replacement approach. It helps manage risks and improves on the previous pipeline replacement strategy. This risk model will be re-run, with updated information, every year to determine if there is a change in the replacement priority and to determine if the previous years selected projects have changed ranking. EBMUD reviews project rankings every two years based on its budget cycle and the implementation order of projects may changes according to risk model results. The annual model review will include updating LOF and COF criteria data sets and evaluate scoring and weighting systems.

Due to the complexity of the risk model and data sets, EBMUD decided to use Innovyze software, *InfoMaster*, to assist in the risk model. This software is compatible with EBMUD's hydraulic model, *InfoWater*. The goal is to eventually link the two models. Both *InfoMaster* and *InfoWater* are GIS based programs that run in conjunction with the ESRI *Arc Map* software. The LDP risk model required an intensive initial effort to collect all required data for each criteria, however, now that the model has been developed it will provide the basis for a justifiable and transparent replacement strategy.

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Understanding Risk and Resilience to Better Manage Water Transmission Systems

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Abstract

The Regional Municipality of Peel, Canada (the Region), a suburb of Toronto, through its growth projections will be tasked with supplying over 2.5 million residential and commercial customers with drinking water over the next twenty years. In response to this, the Region has undertaken a review of its transmission and sub-transmission infrastructure to ensure it can continue to deliver drinking water services that meet its customer's needs. This project consists of undertaking a risk and resilience assessment in order to understand and proactively manage threats and opportunities to key components of the Region's water distribution system and to ensure continued and reliable delivery of water service to its customers. The key focus for the project was to develop a long term strategy to manage and reduce risk through capital improvements and operational planning. In addition, it was necessary to link corporate asset management objectives to risk impacts and resilience enhancements to ensure they are translated into, and support the appropriate business planning processes and life cycle management strategies for the critical assets. The AWWA 7-Step RAMCAP risk management process was used to meet the overall project objectives. Existing and planned protective measures were used to generate alternative options for managing critical asset risks. Risk mitigation options included repair, rehabilitation, replacement, adding redundancy and other operational procedures.

INTRODUCTION

Growth projections indicate that the Regional Municipality of Peel (the Region) could be supplying over 2.5 million residential and commercial customers with a reliable and high quality supply of drinking water over the next twenty (20) years. In response to this, the Region is undertaking a review of its transmission and sub-transmission infrastructure to ensure it can continue to deliver drinking water services that meet customer levels of service (reliability, pressure, quality, etc.). This project was to complete a risk and resilience assessment in order to understand and proactively manage threats and opportunities to the Lake-based water system that form a key component of the Region's municipal services.

The objectives of the project were: (1) Complete a risk and resilience assessment of the Lake-based water system's transmission and sub-transmission water mains; and (2) Develop a risk and resilience management plan for critical water mains within the South Peel system. Additionally, five (5) supporting objectives were identified to meet the needs of the project: (a) Align the project with corporate risk and infrastructure management goals and objectives, (b) Manage stakeholder expectations and project ownership through constructive engagement (c) Build on strategic growth objectives within the Region's business and infrastructure planning processes (d) Use a tailored version of the AWWA J100-10 approach to assess risk and resilience relevant to the Region; and (e) Utilize detailed hydraulic modeling using the Region's model to support inputs to the risk and resilience assessment and project recommendations. A key focus of the project entailed linking corporate asset management objectives to risk impacts and resilience enhancements to ensure they are translated into, and support the appropriate business planning processes and life cycle management strategies for the critical assets.

The Lake-based water system features seven (7) pressure zones, supplied from Lake Ontario by two water treatment plants (WTP). Each zone features pump stations on the East and West side of the systems. These pump stations deliver water to the zones they are located in and also transfer water to the next pressure zone through dedicated transmission mains. The East and West systems are connected through the distribution systems. York Region is also supplied water via the Airport Pump station. The system is shown in Figure 1.

The operational goals of the system are focused on:

1. Minimizing the risk of disruptions to servicing;
2. Maintaining adequate pressures within the system;
3. Maintaining emergency and fire storage within acceptable ranges; and
4. Optimizing electricity use to minimize the annual costs.

Given the configuration of the system, the transmission mains are critical to the transfer of water up the system. The transmission mains in the lower zones are especially critical as all of the zones above a given transmission main are reliant upon it for supply.

APPROACH

The approach for undertaking risk and resilience assessments for the 2012 and 2031 South Peel transmission and sub-transmission systems was built on the AWWA J100-10 standard to include a more holistic approach to managing risks and enhancing

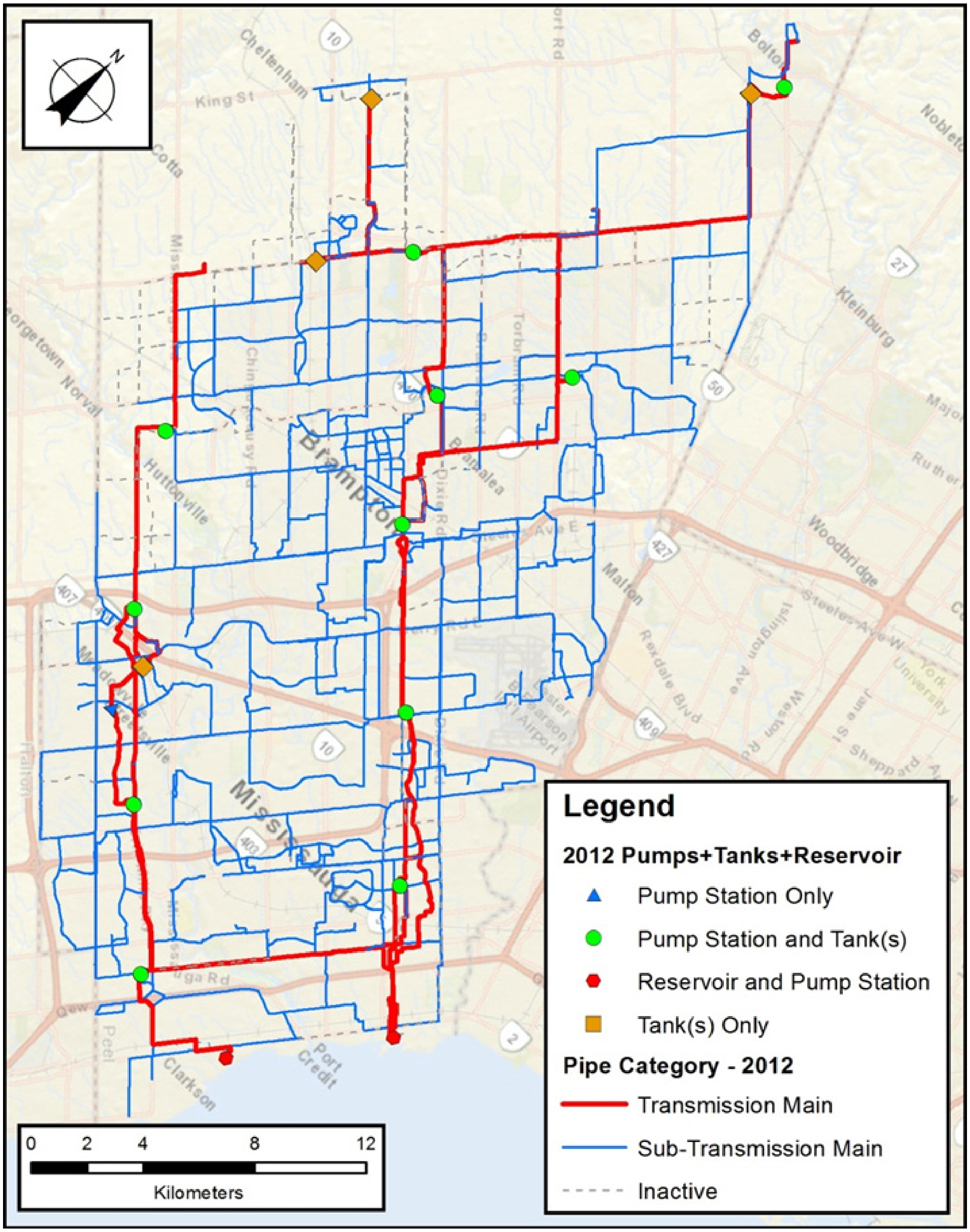

Figure 1. Transmission and Sub-Transmission System.

resilience. In particular, the integrated approach focused on mapping project outcomes into the Region's overall business planning process to support informed decision making, and business cases for capital expenditure, planning and delivery. The risk and resilience assessment will allow for transparent, consistent and repeatable risk (score) outcomes through a systematic and sound assessment approach. Key aspects of the work involved with the Seven-step (Figure 2) Risk Analysis and Management for Critical Asset Protection (RAMCAP) Standard are detailed below. Key resources required to achieve desired outcomes for each of the RAMCAP steps include: (1) Appropriate background data and information; (2) Functional and calibrated hydraulic model; (3) Region staff knowledge and expertise; and (4) Engineer's lessons learned and knowledge from completing similar types of projects.

Ultimately, the Risk for each asset needs to be determined based on the following formula:

Risk = Likelihood x Vulnerability x Consequence x Resilience

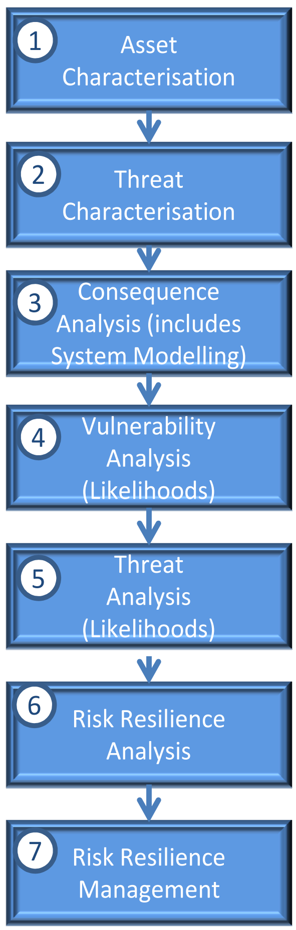

Figure 2. Seven Step RAMCAP Process.

ASSET CHARACTERIZATION

Asset characterization is the first step in the seven step RAMCAP process adopted by the Region and illustrated in Figure 2. The goal of asset characterization step is to divide each of the transmission and sub transmission watermains within the Region's water supply system into segements with unique identifiers. In subsequent phases of the project, risk analysis and management techniques will be applied to each of these segments. This will allow for the identification of critical transmission and sub transmission watermains, as well as prioritization of high risk watermains to be considered for mitigation planning and strategies.

The Region's water supply system consists of approximately seven hundred (700) kilometers or 435 miles of transmission and sub transmission watermains (greater than 600 mm or 24 inches). The basis of the segmentation focused on developing segments by pipe type and geographic location to be commensurate with how the Region plans and budgets for capital water projects, and operates and maintains its watermain infrastructure.

Overall, the segmentation process applied to the Region's water supply system consisted of the following steps:

1. Group segments by each pressure zone
2. Delineate transmission and sub transmission Watermains (within each pressure zone)
3. Divide each transmission and sub transmission watermain into unique segments based on:
   a. Pipe material and diameter;
   b. Proximity or intersection with major highways, railways, creek / river crossings, high density areas, and designated facilities (hospitals, medical centers, and schools); and
   c. Pipe age
4. Assign a unique identifier to each segment based on the pipe material, diameter, age, and GIS code.

The asset characterization process undertaken for the Region's lake based water supply system allows for threat events to be applied to discrete watermain segments within the Region's water supply system. This subsequently allow for further risk analysis and assessment to be completed at a manageable level.

THREAT CHARACTERIZATION

Threat characterization is the second step in the seven step RAMCAP process. The objective of the threat characterization process is to develop a list of threat events that are relevant to the Region of Peel and which may result in failure of the transmission and sub-transmission watermains within the lake based supply system.

In accordance with the RAMCAP methodology, threat events were derived from six (6) general threat event categories including:

- Natural disasters;
- Third party damage;
- Proximity to dangerous sites;

- Operation process / methods;
- Pipe breaks or Physical; and
- Design and Construction

Terrorist threats, drinking water and source water quality threats, and threats associated with climate change are events that were not considered as they are not part of the scope of the risk assessment for this project.

Threat events related to each of the six (6) threat categories were developed based on:

1. Information provided by Region (in the Request for Proposal);
2. The *Deterioration and Inspection of Water Systems: A Best Practice by the National Guide to Sustainable Municipal Infrastructure*; and
3. Engineer's own experiences and knowledge of applicable threats from our design, construction, structural, operational and geotechnical professionals for municipalities in the Greater Toronto Area.

Threat events were initially selected for inclusion based on their relevance to the Region's lake based water supply system. Following this, the Team refined the list of threat events in a workshop. The workshop captured the Region's own familiarity of their lake based water supply system, including operational, maintenance, design, construction, and historical knowledge.

A sample of the final list of threat events that will be applied and evaluated to all transmission and sub-transmission watermains in the risk assessment are presented in Table 1.

Table 1. Sample Threat Event.

Threat Event	Threat Category Two: Third Party Damage	Likelihood Score
2.1	Installations of infrastructure / utilities above existing pipes including traffic poles, hydro poles, traffic signs, manholes, etc., increases external pressure points on a pipe which may lead to weak points	**5**
2.2	Excavating too close to existing thrust blocks can increase the vulnerability of a pipe to soil bearing pressures	**4**
2.3	Installation of utilities (i.e. gas lines, hydro lines, watermain, sewer, etc.) near existing pipes can increase the vulnerability of impact from construction equipment	**4**
2.4	Installation of communication lines near existing pipes can increase the vulnerability of impact from construction equipment	**5**

CONSEQUENCE ANALYSIS

Consequence analysis is the third step in the seven step RAMCAP process adopted. Globally accepted standards and best practices were explored to develop the consequence matrix to allow for transparent, consistent, credible and repeatable results. The Region already developed a triple bottom line consequence matrix for their asset management efforts to date. A 'pair wise comparisons' was also employed to gain further consensus among key project stakeholders. Consequences range from negligible (1) to catastrophic (5) for each of the impacts in the triple bottom line (Table 2). The consequence of each segment failing was determined by geospatial analysis, data analysis, and/or workshop results. In addition to complete the pressure and service demand consequence a detailed hydraulic modeling effort was carried out to support the pressure and service demand impacts. Modeling is presented later in this document.

Table 2. Consequence Matrix.

Triple Bottom Line	**Impacts**
Social	Health & Safety (H&S)
	Pressure
	Service Demand
	Reputation
	Quality of Service
	Cost of Restoration
Financial	Financial Impact
	3rd Party Claims / Litigation / Fines
Environment	Regulatory
	Physical Environment / Community

VULNERABILITY ANALYSIS

Vulnerability analysis is the fourth step in the seven step RAMCAP process adopted. The aim of a vulnerability analysis is to assess the extent to which transmission and sub-transmission segments within the Region's lake based supply system can withstand the potential theat events as defined in the AWWA J100-10 guideleines. In other words, vulnerability analysis is based on the assumption that a threat event will materialize and impact a segment, given the threat event occurs.

Vulnerability criteria are typically associated with asset attributes, design parameters, operating conditions and site data. These criteria are used to supplement the likelihood evaluation of a segment being able to withstand a threat event, or a threat event being able to overcome the defenses of each segment.

Vulnerabilty Scale(s)

In line with the RAMCAP process, the vulnerability of a segment or asset can be estimated as a range on a 'vulnerability scale'. Vulnerability scales are usually expressed as ratios, percentages, decimals or as a single point estimate. From a process prospective, a scale convenient and or familiar to the user is preferred since evaluations will rely on expert judgment. For the Peel lake based risk assessment, a percentage based range (0%, 10%, 25%, 50%, 75%, 100%) was proposed for evaluation.

Vulnerability Criteria

Vulnerability criteria were initially derived from four (4) asset categories including pipe condition, pipe location, original construction details, and operating conditions. In a workshop seven (7) vulnerability criteria including pipe age, pipe material, pipe location, operating pressures, soil types, restraining joints, and bedding material were identified. At a later workshop the project team came to a consensus to develop vulnerability criteria specific to each threat event.

Sample vulnerability criteria for a threat event is presented in Table 3.

Table 3. Threat Category: Soil Environment.

Threat Event	Vulnerability Criteria	Vulnerability Weighting
- *Corrosive soils can cause and / or increase the corrosion rate of a pipe; or* - *Pipe leakage can lead to the erosion of surrounding pipe bedding and therefore can lead to an increase in the stresses acting on a pipe (Some soils are prone to large volume changes in the presence of moisture, leading to an increased loading on a pipe); or* - *Some backfill materials are vulnerable to freezing. This could lead to a thermal stress on the pipe.*	Pipe is installed in 'well drained' soils	0.1
	Pipe is installed in mainly sandy loam or silt soils with 'imperfect' drainage or loam soils with 'poor drainage' or 'organic' soils with 'very poor drainage'	0.25
	Pipe is installed in mainly clay loam soils with 'poor' or 'imperfect' drainage or is not classified	0.50
	Pipe is installed in mainly clay soils with 'imperfect' drainage	0.75
	Pipe is installed in mainly clay soils with 'poor' drainage	1.0

THREAT ASSESSMENT (LIKELIHOOD)

The Threat Assessment, or determining Likelihood, is the fifth step of RAMCAP. The Region's corporate likelihood matrix (developed previously by the Region) is presented in Table 4. Referencing the Region's likelihood matrix for this project ensures alignment with the organization's overall risk tolerances, and allows for comparison of risks across Regional Departments.

Table 4. Region of Peel – Likelihood Matrix and Descriptors.

Likelihood	Descriptor	Score
Rare	An occurrence /situation is not likely to occur within 20 years	1
Unlikely	An occurrence / situation is not likely to occur within 10 years but possibly within 20 years	2
Possible	An occurrence / situation might occur within 10 years	3
Likely	An occurrence / situation might occur within 2 years	4
Almost Certain	An occurrence / situation that is happening or immanent and / or will probably occur within 1 year	5

The Region's likelihood matrix was referenced to assign likelihood estimates for the probability of occurrence of each threat events. The Likelihood of each threat was determined by geospatial analysis, data analysis, and by workshop. Table 5 presents a sample Likelihood scoring for a threat event.

Table 5. Likelihood Example Scoring.

Threat Event	Design/Construction	Likelihood Score
5.1	Poor installation practices can damage pipe coatings, or cause fractures leading to weak points on the pipe. (Cyclic stresses will accelerate crack propagation and failure).	4
5.2	Defects in pipe walls caused by the manufacturer can increase vulnerability to failure.	3
5.3	Improper bedding may lead to increases loadings / stresses on a pipe and eventual premature failure.	4
5.4	Groundwater may be aggressive to certain pipe materials.	5

USE OF HYDRAULIC MODELING

A critical piece of the consequence analysis and also resiliency (discussed later in this document) is the use of the Region's calibrated hydraulic model. The model was calibrated to complete both steady state and extended period simulations (EPS) for current conditions (2012) and future conditions (2031). The Team determined the consequence of each segment impacted by a threat (shutdown or closed) would have on the entire distribution system for a selected period of time using the EPS model. The Region wants to maintain an operating pressure of 40 PSI at all times in areas with demands. Forty eight (48) hours and 7 days were selected for the analysis periods for sub-transmission and transmission mains respectively (this is the approximate time needed for the region to repair these segments). In addition, if the closing of a segment caused a demand shortfall the total population impacted was also calculated. Tables 6 and 7 summarize the pressure and demand consequence scores.

Table 6. Pressure Consequence Scores.

Critical Node Pressure	Vulnerability Pressure Consequence Score
≥ 40.0 psi	1 (Negligible)
35.1 to 39.9 psi	2 (Low)
30.1 to 35.0 psi	3 (Moderate)
20.1 to 30.0 psi	4 (Severe)
< 20.0 psi	5 (Catastrophic)

Table 7. Demand Shortfall Consequence Scores.

Service Demand Consequence Score	Number of People Affected
1 (Negligible)	0 to 99 persons
2 (Low)	100 to 999 persons
3 (Moderate)	1,000 to 9,999 persons
4 (Severe)	10,000 to 49,999 persons
5 (Catastrophic)	≥ 50,000 persons

The model was run with each segment closed for the selected time period and the impact assessed and scored. Each segment was then assigned a pressure and demand consequence score. The analysis was performed for both 2012 and 2031. Samples of the results are shown in Table 8 and Figure 3.

RESILIENCE

Resilience is defined by AWWA J100-10 as the following:

Resilience = Duration x Severity x Vulnerability x Likelihood

For the Region duration is defined as the time without water if there is demand shortfall in days. Severity is defined as the volume of the demand shortfall in gallons per day (gpd). Vulnerability and Likelihood were taken from the previous analysis.

In reviewing the results the lower the Resilience score the more resilient a segment is to the threat. For example, if a segment is closed and the model run shows there is no pressure impacts (all pressures greater than 40 PSI) or no demand shortfalls then the segment is very resilient (score is 0).

Table 8. Sample Pressure Consequence Scores.

Segment	Pipe Closed	Number of Pressure Violations	Average Pressure Violation (psi)	Maximum Pressure Violation (psi)	Critical Node	Pressure Supplied at Worst Node (psi)	Criticality Pressure Consequence Score
R48	525874	115	1.1	2.6	6566764	37.4	2 (Low)
H28	526381	5	0.5	0.9	1601941	39.1	2 (Low)
G11	746048	5377	>100	>100	608884	<=0	5 (Catastrophic)
V02	755478	310	3.6	8.1	606767	31.9	3 (Moderate)
D74	WM-595	1446	>100	>100	1601941	<=0	5 (Catastrophic)

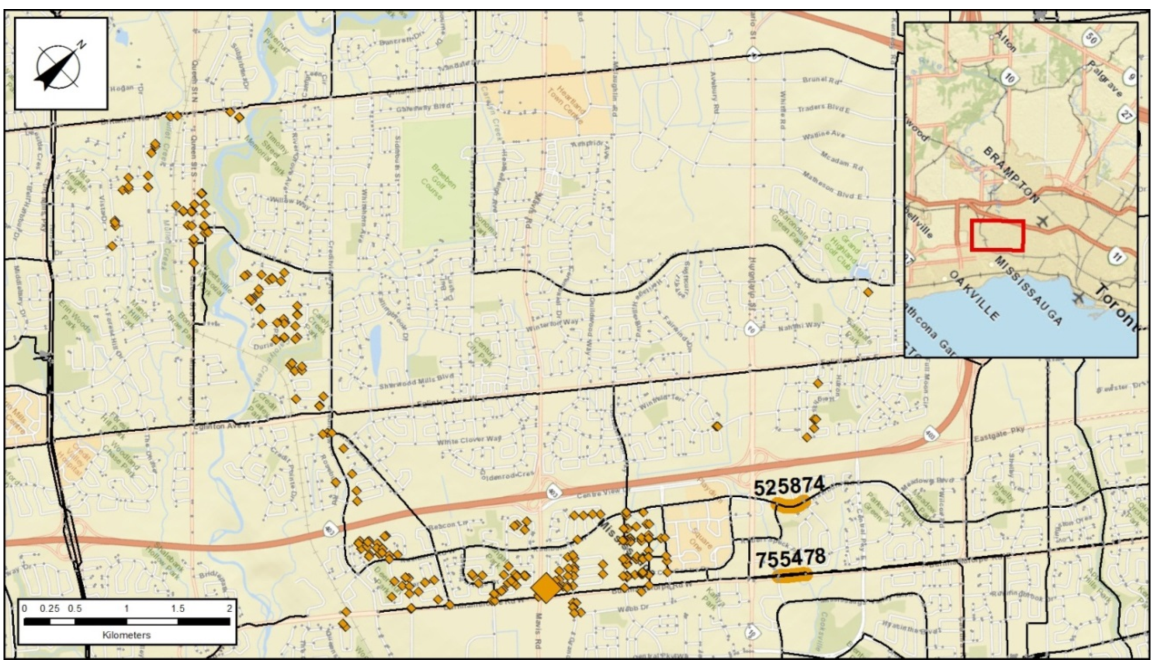

Figure 3. Sample Pressure Consequence scores for Segment V02 (pipe 755478).

Modeling results were critical to this analysis. These data are not explicitly available from the model but can be extracted and calculated for each segment. This was done for both the 2012 and 2031 models.

RISK ASSESSMENT

The sixth step is to determine the overall Risk for each segment in the system. For each asset a Risk score was determined based on Formula 1 for each threat event. Then the risk score for each threat event is summed to get the Total Risk Score (TRS) for each segment. An example is shown in Table 9.

Table 9. Highest Risk Water Pipe Segments: Zone 1-2012.

ID	TRS	R01	R02	R03	R04	R05	R06	R07	R08	R09	R10	R11	R12	R13	R14	R15	R16
T29	140	0	1	14	9	14	7	9	18	5	1	18	14	7	1	14	14
U28	119	2	1	12	8	13	6	8	2	5	1	16	12	3	1	13	16
T19	116	3	1	5	2	1	7	9	18	1	1	18	14	4	1	14	18
J68	109	0	1	12	8	12	6	8	2	1	1	16	12	6	1	12	16
E26	106	0	1	12	2	12	6	8	2	1	1	16	12	6	1	12	16
D11	106	2	4	11	7	11	6	7	1	4	1	4	11	11	1	11	14
U35	101	2	4	11	1	11	6	7	1	4	1	14	1	11	1	11	14
V30	100	2	1	11	7	11	6	7	1	1	1	14	1	11	1	11	14
E08	99	0	1	10	7	11	5	7	1	4	1	14	10	3	1	11	14

Now that the Total Risk Score is calculated for each segment they can be plotted to determine the assets at highest risk. Figure 4 is a plot of the Region's segments for the 2012 scenario. What is important to understand is the level of business risk exposure (BRE) an organization is willing to bear. BRE is the product of probability of failure and the consequence of failure for each asset. However, it is more clearly communicated a graph. In the figure below is a line drawn to show a possible BRE for this project for illustrative purposes. Segments (assets) above this line (Zone 1) are above the organizations risk tolerance and should be the focus of mitigation efforts to lower the overall risk. Assets below the line (Zone 2 and Zone 3) are to be managed so that their risk does not change and cross the line. You will note some assets do not fall below the line at all (Zone 4) to the far left of the figure. These assets are effectively managed to complete failure as their consequence of failure is not great enough to manage their risk.

RISK MANAGEMENT/MITIGATION

The seventh and final step of RAMCAP is to do a Risk Management/Mitigation Plan. A detailed risk and resilience management plan will be developed with an emphasis on optimizing Region resources together with benefits and costs. Existing and planned protective measures will be used to generate alternative options for managing critical asset risks. Risk mitigation options can include repair, rehabilitation, replacement, adding redundancy, sub-metering with remote telemetry. Constructive engagements and consultations with key Region staff and industry experts will be employed to develop alternative solutions that meet the Region's risk tolerances and business needs. Benefit cost ratio methods supplemented by additional economic techniques for project comparisons will be utilized to identify the 'best' options and

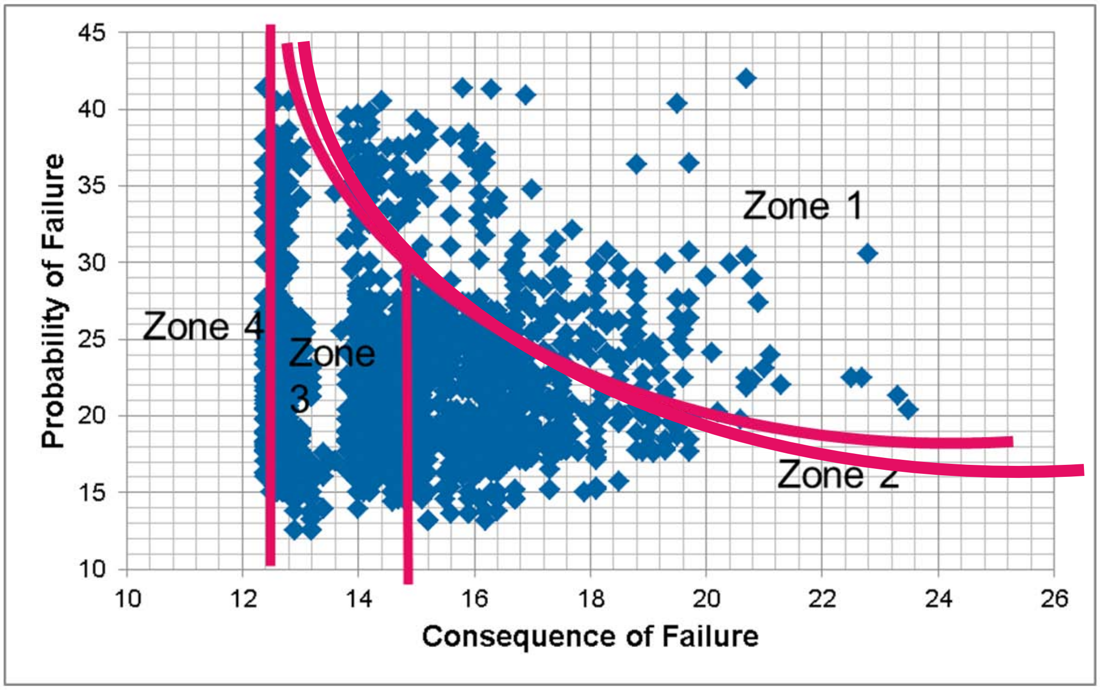

Figure 4. Risk Assessment-2012 Risk Results.

support the Region's decision making processes and business case needs. Detailed monitoring and mitigation plans will be developed for selected options taking account of current operational and maintenance philosophies and audit processes. This will allow the Region to identify 'quick wins' for this project through comparison of proposed alternative options with the current capital implementation program and master plan. The overall process is illustrated in Figure 5.

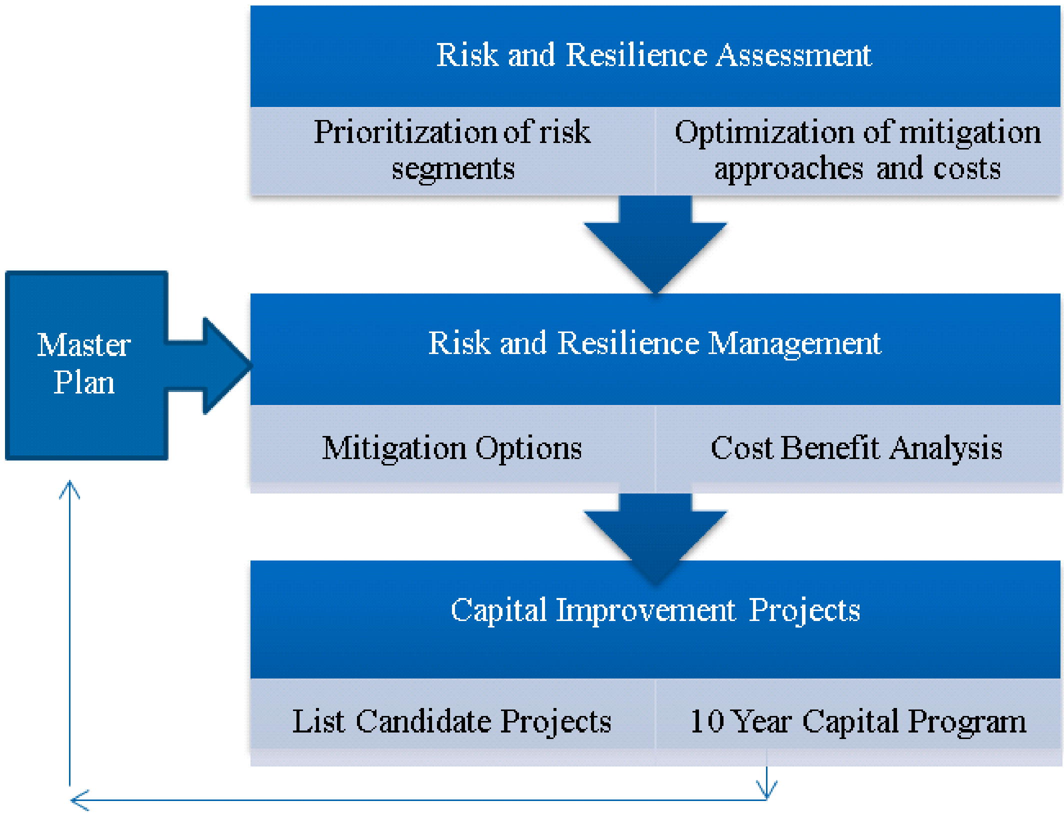

Figure 5. Mitigation Options and Cost Benefit Analysis.

CONCLUSION

At the completion of this project the Region had a more complete understanding of the assets that are at the highest risk to the overall transmission and sub-transmission system. The use of the AWWA 7-Step RAMCAP risk management process provided an unbiased review of all assets in the study that aligned with corporate asset management objectives. For the assets of highest risk a mitigation plan was developed for each one that may include operational changes, emergency response plans, and/or capital improvements. In addition, the Region now has a system available to them to continue to revise the risk analysis as mitigation plans are implemented and/or more information that impact the risk of an asset becomes available.

Shifting the Paradigm from Replacement to Management

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Abstract

Cobb County-Marietta Water Authority (CCMWA) is the second largest drinking water supplier in Georgia, providing service to nearly 800,000 people. Two of CCMWA's key objectives are to be financially viable and to reduce vulnerabilities by improving redundancy and implementing a comprehensive asset management program. A large portion of CCMWA's large-diameter pipeline inventory is made up of PCCP. In 2012, CCMWA was in a similar situation to many predominant PCCP users; past failures on these critical assets had led to the decision to replace the majority of PCCP assets to avoid the risk of future failures. To date, CCMWA has replaced 46% of its PCCP inventory through capital improvement programs and emergency responses. Since 2012, CCMWA has been proactively managing the remaining PCCP within their inventory through a comprehensive condition assessment program. This paper will discuss the various methods utilized by CCMWA to manage their PCCP inventory including the financial and operational impacts of these approaches.

INRODUCTION AND BACKGROUND

With infrastructure aging across the United States, utility operators and end users alike are seeing an increased number of water pipe failures. While these failures are most commonly on small pipes – causing only minor disruptions – large-diameter mains do fail, resulting in major delays and enormous repair bills.

As the second largest drinking water supplier in Georgia, providing vital service to nearly 800,000 people through thirteen wholesale customers, Cobb County-Marietta Water Authority (CCMWA) is no stranger to the struggles of maintaining a large diameter inventory. With two award-winning water treatment plants and over 200 miles of large-diameter transmission mains, CCMWA can deliver up to 158 million gallons per day. Although CCMWA's large-diameter mains are constructed using a variety of pipe materials, the majority of the transmission main inventory consists of Prestressed Concrete Cylinder Pipe (PCCP). Since its inception in 1951, CCMWA has installed nearly 160 miles of PCCP with its oldest continuously operating pipeline installed in 1958. An overview of the transmission and raw water system is shown in Figure 1.

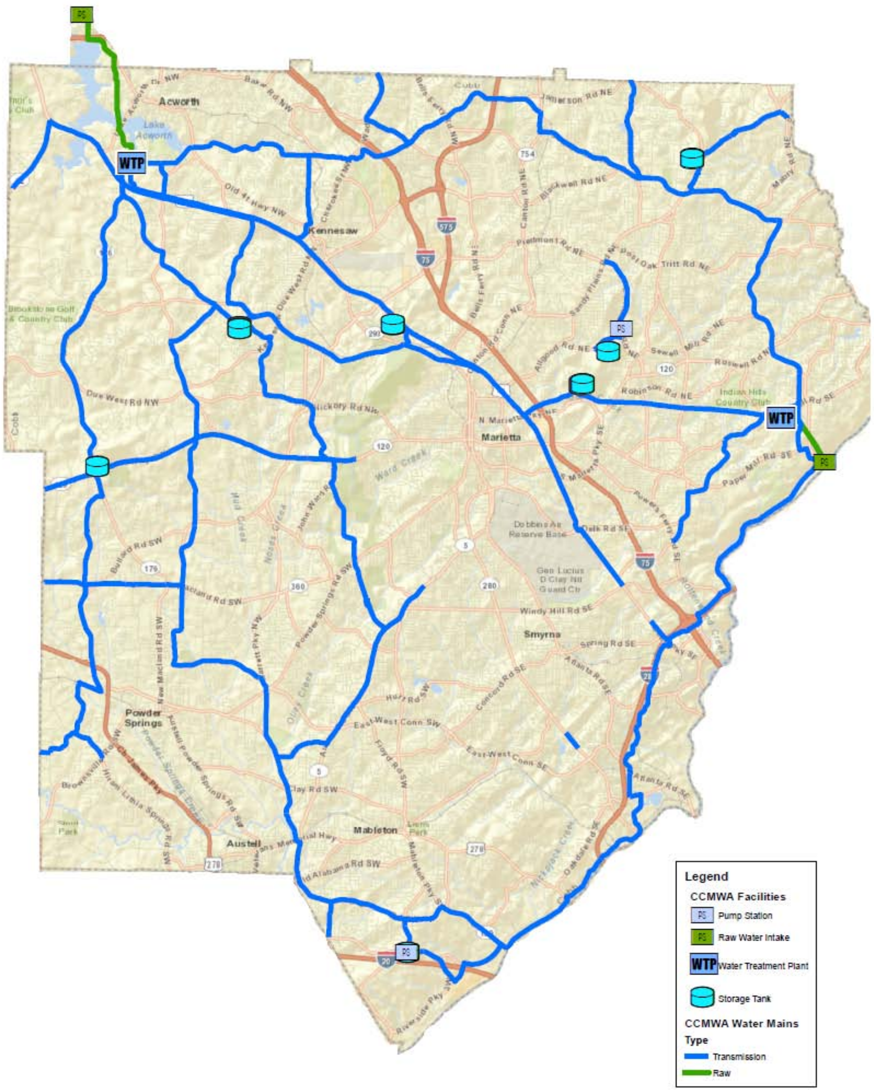

Figure 1: CCMWA Transmission and Raw Water System Overview

PCCP has been used for large diameter water transmission and distribution mains since 1942 [3]. A typical lined cylinder PCCP (LCP) consists of a concrete liner, a concrete core, a thin steel cylinder, high strength steel prestressing wire and a mortar coating. A cross sectional view of LCP is shown in Figure 2. The concrete core and prestressing wire are the main structural components, while the steel cylinder acts primarily as a water barrier. The prestressing wire produces a uniform compressive force in the core that offsets the tensile stresses developed in the concrete from the internal water pressure. A mortar or concrete coating surrounds the prestressing wire, embedding the wraps in an alkaline environment to protect them from external corrosive influences (such as acidic soil and groundwater). The coating also provides protection from physical damage.

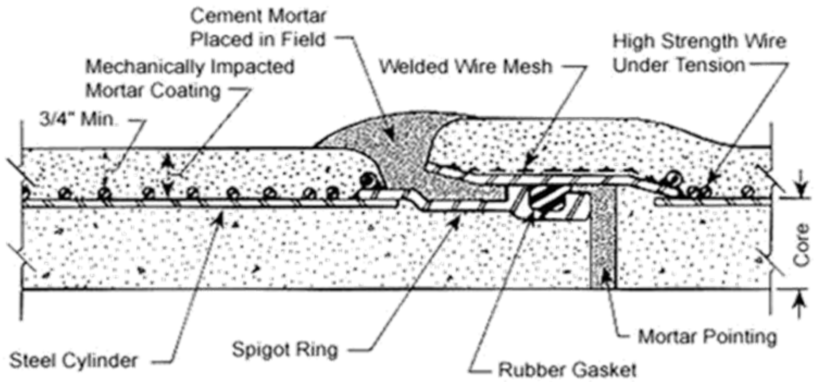

Figure 2: Cross-sectional view of LCP [3]

PCCP design and manufacturing standards have gradually developed since 1943, with the first tentative consensus standard for PCCP being approved by the AWWA in 1949 [1]. The initial structural design requirements for the manufacturing of PCCP tended to be conservative, with high factors of safety [1,5]. As experience with using this composite pipe grew, understanding of the behavior of PCCP increased, and advances in material sciences were achieved, the structural design of PCCP was changed to reduce the cost of manufacturing. Increases in the applied tensile strength of the wire that occurred during manufacturing in the late 1960s and early 1970s reduced the amount of prestressing steel wire required and allowed for the use of smaller diameter wire. This resulted in what appeared to be a more efficient design and cost-effective manufacturing process.

These practices culminated in the 1970s, when pipes using an even stronger wire, and other cost saving measures, were manufactured. This wire was produced by using a loophole in the ASTM and AWWA standards, which did not define a maximum tensile strength. All classes of prestressing wire are susceptible to external corrosion and other failure modes. However, this wire is also sensitive to hydrogen embrittlement and dynamic strain aging effects. Pipes from this era started experiencing a high rate of premature failures, primarily related to the new standards and manufacturing processes. Consequently, the engineering standards for PCCP began to improve, resulting in better manufacturing standards.

Several causes for PCCP failure have been observed by CCMWA including poor quality of mortar coating, poor quality of prestressing wire, a corrosive environment, construction damage, and delamination of coating. Most PCCP failures result from a breakdown of corrosion protection leading to corrosion or hydrogen embrittlement of the prestressing wire. This causes incremental wire break damage that grows with time until the pipeline eventually ruptures. As each wire wrap breaks, the individual pipe's strength is incrementally reduced.

The American Water Works Association Research Foundation (AWWA-RF) completed a study on the modes of failure experienced in over 500 sections of PCCP [1]. Category 1 failures were characterized as catastrophic failures and leaks of the main. Category 2 failures were defined as significant deterioration or structural weakness discerned by various inspection techniques including visual/sounding and electromagnetics. The Category 1 (Blue) failure rate for the pipe sections manufactured in 1972 to 1978 is significantly higher due to material quality and design methods utilized during that time. Figure 3 details the probability of failure based on the year of production. Figure 4 details the miles of PCCP installed at CCMWA by production year and it is evident that the bulk of the inventory falls into the higher failure rate categories.

Figure 2 gives evidence to the transformation of Cobb County in the 1960s and 1970s from rural to suburban as the county population grew by nearly 85% throughout the decade. CCMWA was in a similar situation to many predominant PCCP users as past failures on these critical assets had led to the decision to replace the majority of PCCP assets to avoid the risk of future failures. To date, 46% of CCMWA's PCCP inventory has been replaced through capital improvements and emergency response.

In 2009, amidst a nation-wide recession, a strategic plan was developed by CCMWA to address the challenges they faced including drought, increased competition for water supplies, a growing regulatory burden, aging infrastructure, changing workforce, and higher stakeholder expectations regarding levels of service, efficiency, and environmental responsibility. As part of the strategic plan, two of CCMWA's key objectives are to be financially viable and to increase reliability through a reduction in vulnerabilities by improving redundancy and implementing a comprehensive asset management program.

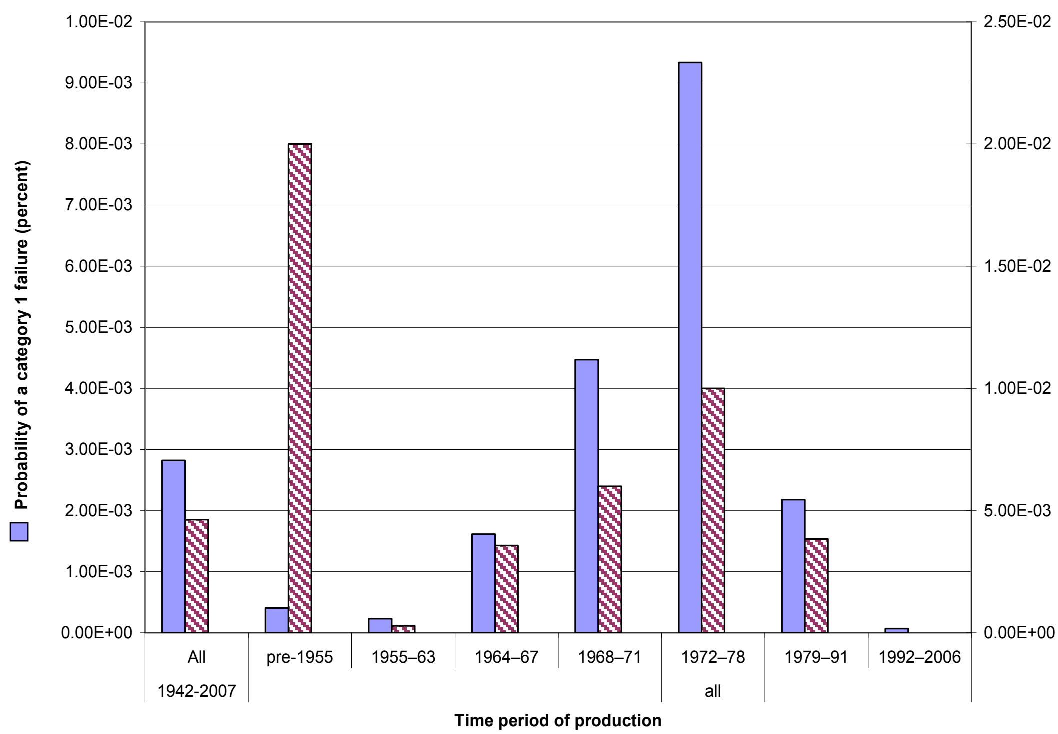

Figure 3: Failure of PCCP by Pipe Vintage [1]

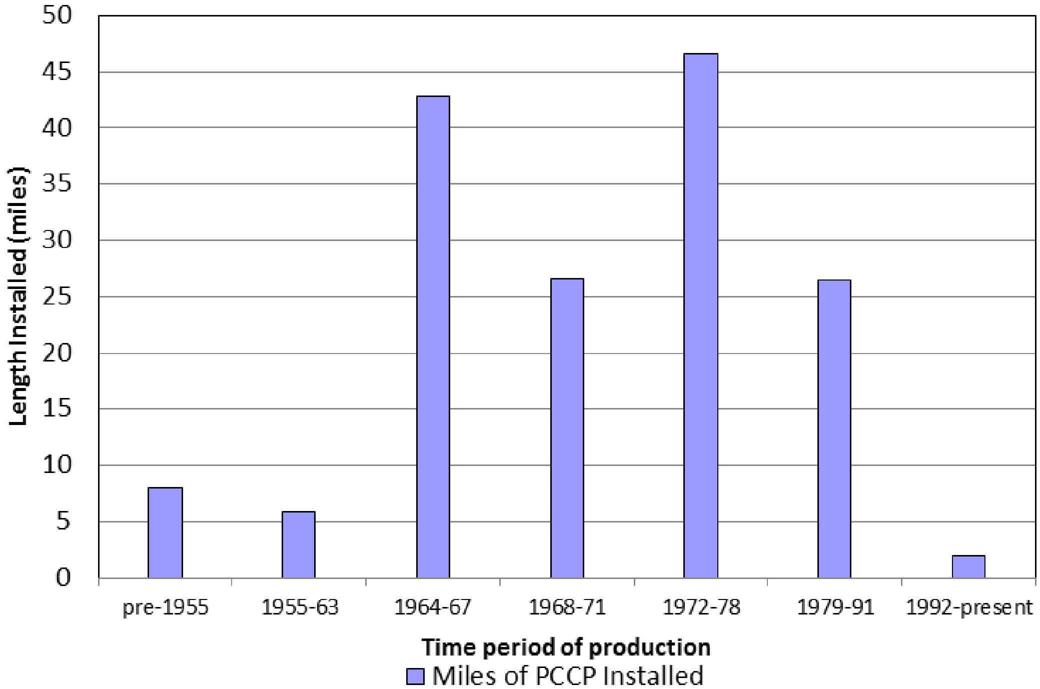

Figure 4: Miles of PCCP Installed at CCMWA by Year

FINANCIALLY VIABLE MANAGEMENT OPTIONS FOR PCCP

The three most common methods of pipeline management include doing nothing and wait for failure, capital replacement of the entire asset, or implementation of pipeline asset management strategies. The do nothing approach is deemed unacceptable by CCMWA as the consequence of failure is too great throughout the entire PCCP inventory; therefore, a capital replacement strategy was implemented in the 1990s to purge the inventory of all PCCP, beginning with the class type installed in the 1970s which was more susceptible to failure.

To date, 46% of the PCCP inventory has been replaced. Replacing large sections of pipeline was found to be neither financially nor logistically feasible. Large-scale replacement programs are also unnecessary based on industry research, which confirms that pipe deterioration is not uniform and is related to localized problems. By making the decision to replace long stretches of pipeline, operators could be replacing assets that are in like-new condition. Advances in inspection methods have indicated that typically less than 5% of a pipeline experiences any level of distress, and less than 1% of the pipeline is affected by distress that requires immediate action [2]. The use of condition assessment to manage PCCP mains has been widely adopted by the water industry and has a proven track record of identifying and averting PCCP failures. PCCP operators continue to use the various condition assessment methodologies combined with sound engineering analysis to effectively and safely manage their assets.

Given that there are tools and techniques available to investigate PCCP and find individual pipes with damage, an evaluation of costs can determine what management strategy would be most effective for a pipeline. A financial evaluation based on the cost of capital replacements compared to regular condition assessment inspections on the remaining PCCP water mains in CCMWA's inventory shows that the average pipeline can be managed for approximately 15% of the capital replacement costs when extended over 25 years with an escalation rate of 2.5% and a discount rate of 5.5%. Figure 5 and Table 1 summarize the 25-year results based on the cost of management in 2013 dollars.

Table 1. Financial Comparison of Management Option Costs

Management Option	Net Present Value of Cost Unti1 2038 ($M, USD)	Percent of Capital Replacement Cost
Capital Replacement	$82.0	N/A
Assess & Address	$11.3	13.7%

This evaluation compares the cost of managing the existing PCCP assets against the cost of capital replacement. For simplicity's sake, only CCMWA's 36-inch PCCP inventory was utilized as the majority of the system consists of this diameter. The cost of analysis of the assets was derived from a simple analysis of historical pipeline information, data collected as part of over 600 miles of PCCP condition assessment, financial data for failure costs, and estimated replacement costs for the pipelines. All of these costs were evaluated for a 25-year future projection.

The financial modeling for the capital improvement program methodology started at the beginning of a replacement investment cycle. The first time investment was arrived by balancing the cost of the new pipe material, the cost for removing the old pipe, and other associated engineering and construction costs. CCMWA estimates $615 per foot of pipe for material and construction costs while an additional 15% was added to this cost for engineering fees. The assumed investment cycle is 20 years. It was determined there would be a regular yearly operation and maintenance cost under the program and there would be a less regular routine maintenance cost that would be a factor of the pipeline's current condition and occur at a regular interval based in this condition. Initial maintenance costs include leak detection of the new pipeline beginning within seven years of installation and repeated assessment every seven years thereafter.

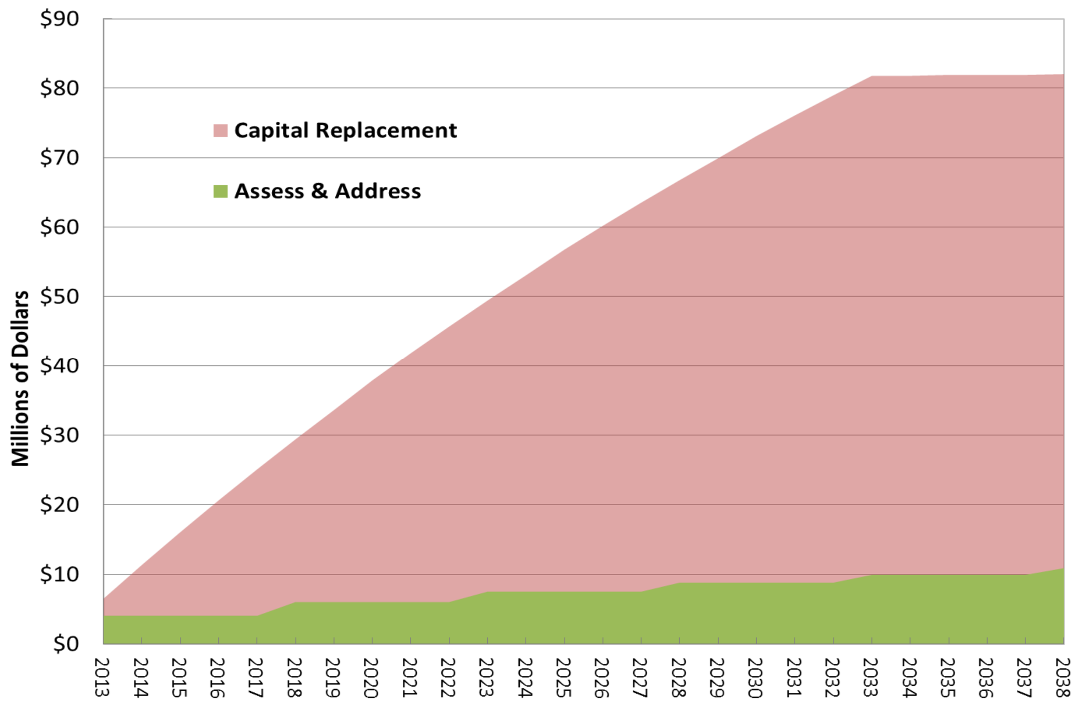

Figure 5: Financial Evaluation of CIP vs Condition Assessment

The modeling for the Assess & Address management strategy considered an input for a typical inspection period of five years using standard pricing for non-tethered inspection technologies. No dewatering costs for inspection are included in the financial analysis. Cost for repairs is based on a 1% distress rate upon first inspection and a 0.5% distress rate upon each re-inspection period which is the typical distress rate identified within this pipe material [2]. It should be noted that it is important to evaluate the full risk of a pipeline using both the likelihood and consequence of failure to arrive at the optimized management strategy. Because consequence of failure is utility dependent with a high degree of subjectivity, this analysis focuses on the likelihood of failure at this time.

INCREASING RELIABILITY THROUGH PROACTIVE PIPELINE MANAGEMENT

In 2012, CCMWA decided to manage its critical PCCP assets using condition assessment and engineering analysis, an approach widely adopted by proactive PCCP owners. In 2013, CCMWA completed its first condition assessment to identify structural deterioration on its PCCP. The project focused on the 42- and 30-inch main, shown in Figure 6, which had previously failed to ensure that it could be safely operated.

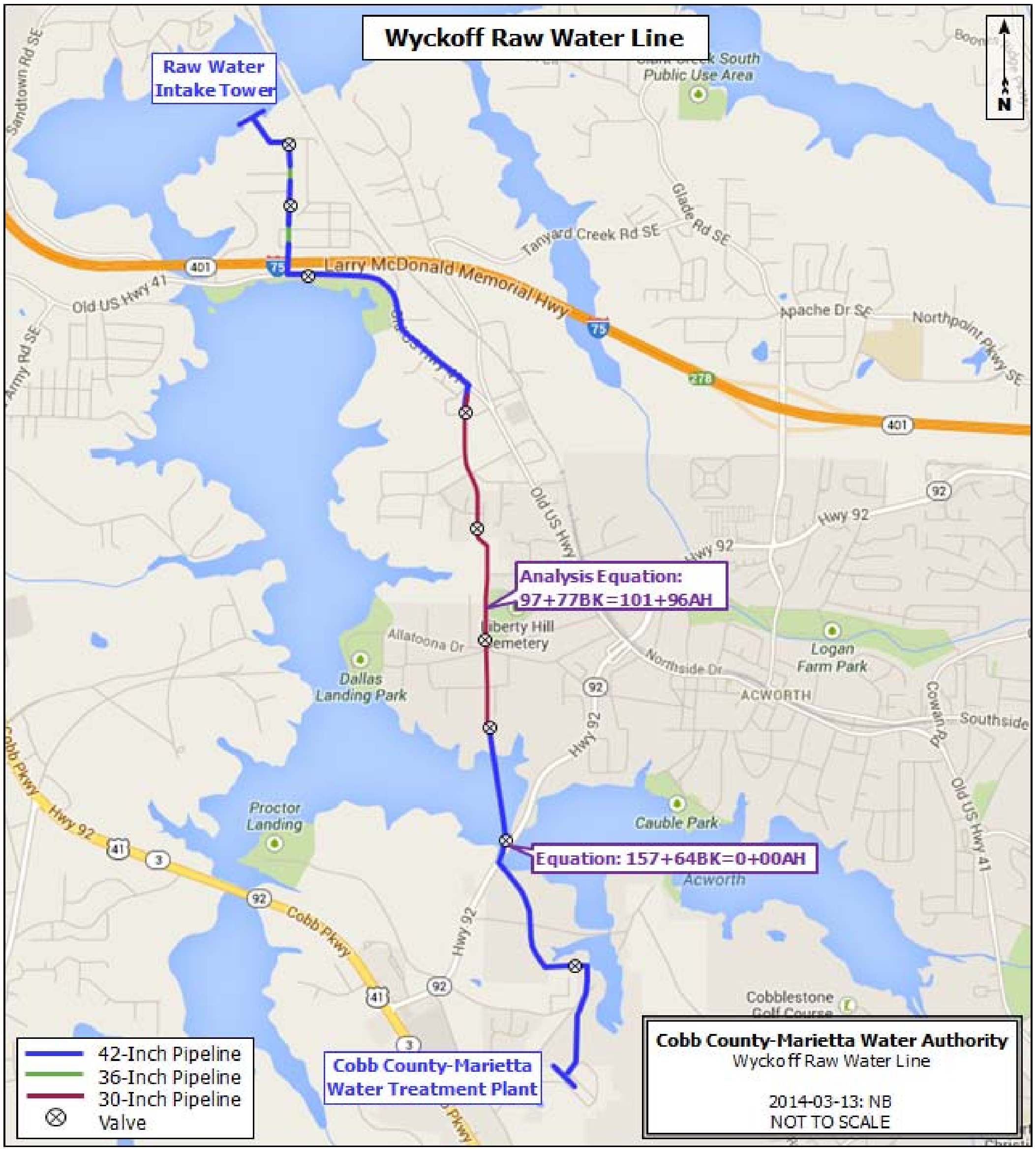

Figure 6: Map of the Wyckoff Raw Water Line and the corresponding inspection limits.

The assessment featured two inspections – a prescreening leak and gas pocket survey and inline electromagnetic inspection – on roughly four miles of 30- and 42-inch PCCP Raw Water Line. The subject pipeline acts as a redundant supply line from Lake Acworth to the Wyckoff Water Treatment Plant. The project also included engineering evaluations including structural analysis and remaining useful life evaluations to make management and renewal recommendations.

For the prescreening survey, CCMWA used SmartBall® leak detection, a free-flowing tool that identifies the acoustic anomalies associated with leaks and gas pockets in large-diameter pipelines. Completing a prescreening leak and gas pocket survey is a prudent approach for operators of any pipe material, since leaks are often a preliminary indication of a failure location, especially in metallic pipelines. For

PCCP, leaks are usually located near the pipe joint, which is also a common failure area on PCCP. The inspection did not identify any leaks nor pockets of trapped gas.

For the more detailed structural evaluation, the PipeDiver® electromagnetic inspection platform was used. The tool uses electromagnetics to identify broken prestressing wires, which are the primary structural component in PCCP. As sections of PCCP begin to deteriorate, the prestressing wires begin to break which weakens the pipe and makes it more likely to fail. Identifying broken wires is the most effective way of determining the condition and preventing failures in PCCP.

By completing an EM inspection on the PipeDiver platform, CCMWA was able to determine the baseline condition of the pipeline while it remained in service – a major benefit for operators who cannot remove mains from service to complete internal inspection.

For CCMWA, the inspection identified ten (10) pipe segments, less than 1% of the pipeline that indicated broken prestressing wire wraps. On average, PCCP inspections across the country indicate approximately 4% of the pipe segments indicate any level of damage. This confirms that the majority of PCCP is in good condition, with only a small number of pipe sections in need of immediate renewal.

Locating and renewing even one pipe section can help utilities maintain reliable service and avoid an expensive pipe failure. Based on an *American Water Works Association* study, the cost of mitigating a single large-diameter pipe failure can range from US$500,000 to US$1.5 million.

Beyond the prescreening and structural inspections, CCMWA was able to identify limitations in its potable water system through the planning portion of the project. The inspected pipeline carries raw water to the Wyckoff Treatment Plant. In order to ensure that the main was being operated safely within its limits, a hydraulic study was completed by the operators. This study found that the 30-inch section of the pipeline, which had failed previously, was incapable of supplying the treatment plant's required operating flow rate and maintain safe operating pressure within the system. Operating the pipeline under these required conditions would place the pipeline at a higher risk of failure. Based on the study, it was recommended that the approximate 1-mile of 30-inch PCCP be replaced to handle existing and future operating condition requirements of the treatment plant. CCMWA was pleased with the discovery, as it allowed them to make scientifically defensible decisions about their 30-inch PCCP main and pumping station while contributing to the prevention of future pipe failures.

For managing the remaining 42- and 36-inch portions of the pipeline, re-inspection analysis was performed in order to cost-effectively minimize the risk of failure while maximizing the value of the pipeline. A model was developed to assist in the development of long-term management strategies. This model simulates the deterioration of a pipeline and can assist in determining investigation techniques, re-inspection intervals, and capital replacement needs by incorporating inspection data, structural analysis, and design information while leveraging over 600 miles of pipeline condition assessment data of PCCP at various locations throughout the US. This model is an extension of the original Markov process proposed by Kleiner in *Aqua-Journal of Water Supply: Research and Technology* [4].

The Markov deterioration model is completed from the date of distress initiation to the date of inspection so that the degradation rates can be computed based on two known condition; the assumed original condition and the current condition based on inspection data. The rate of pipe developing distress in the Wyckoff Raw Water Line was found to be low as after a nearly 50-year service life, less than 1% of the PCCP sections display electromagnetic signatures consistent with wire wrap damage. This low rate of distress initiation and deterioration result in a minimal risk of pipeline failure and a re-inspection was recommended within eight years in order to determine if pipeline deterioration has accelerated.

CONCLUSIONS

By managing its PCCP assets, CCMWA has been able to identify limitations in its system that have allowed for intelligent capital planning and long-term pipeline management.

As a large portion of CCMWA's large-diameter pipeline inventory is made up of PCCP with the majority of its PCCP consisting of a wire class found to be susceptible to failure, capital replacement of the PCCP inventory was implemented. To date, 46% of its PCCP inventory has been replaced with approximately 73 miles still remaining in service.

In 2009, CCMWA implemented a strategic plan that focused on financial viability and system reliability and in 2012 a financial evaluation based on the cost of capital replacements compared with PCCP management (inspection, repair, re-inspection, and repairs) for the PCCP in CCMWA's inventory indicated that the pipelines can be managed for approximately 15 percent of the capital replacement costs when extended over 25 years using a net present value calculation.

Since 2012, CCMWA has been proactively managing the remaining PCCP within their inventory through a comprehensive condition assessment program.

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Baltimore's First Step towards Advanced Pipeline Management

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Abstract

Baltimore Metropolitan Water District (BMWD), which provides water to 1.8 million people, decided to undertake a risk-based prioritization for the network of large diameter mains that is the backbone of the system. This includes 86 miles of PCCP. The goal of the prioritization is to provide the BMWD with a transparent and defensible action plan to minimize the risk of catastrophic failure associated with PCCP mains. The prioritization was conducted by grouping pipelines into a logical approach for the overall asset management strategies the BMWD is currently implementing. This desk-top prioritization effort is a planning level tool, based on the current knowledge of the system that will be used to guide the inspection sequence. The resulting model can be easily updated on a semi-annual or annual basis as actual field data is collected and the true knowledge of the system improves. This paper will present the methodologies used to conduct the prioritization and the results of the prioritization demonstrating the importance of a cohesive plan.

INTRODUCTION AND BACKGROUND

The Baltimore Metropolitan Water District (BMWD) provides water to approximately 1.8 million people in Baltimore City, Baltimore County, Anne Arundel County, and Howard County. In order to accomplish this, the BMWD maintains 3800 miles of water mains. Of these 3800 miles, approximately 145 miles (3.8%) are considered large diameter mains (36-inch and larger). These form the backbone of the system. Eighty six miles (60%) of the large diameter mains are constructed of Prestressed Concrete Cylinder Pipe (PCCP). Figure 1 is a map showing Baltimore's location in the Mid-Atlantic and Figure 2 depicts the location of all large diameter mains.

Figure 1. Baltimore's Location

In order to protect these critical assets, the BMWD will begin an aggressive 10 year project of inspection, repair or replacement, maintenance, and monitoring of the PCCP inventory. Previously, in 2007 and 2010 the BMWD conducted a pilot inspection program to determine appropriate condition assessment, repair, and monitoring techniques. The planned program will build on the information collected and analyzed during that project. While this prioritization did an excellent job documenting the PCCP inventory it did not take into account AFO monitoring. By creating a cohesive, holistic long term plan, the BMWD will have the ability to inspect relatively long lengths of pipe via one mobilization. An optimized schedule of inspections and subsequent monitoring system deployment will provide maximum value by minimizing shutdown and dewatering logistics, increasing efficiency, providing a technologically superior pipeline management approach, and minimizing the risk of failure associated with the PCCP inventory.

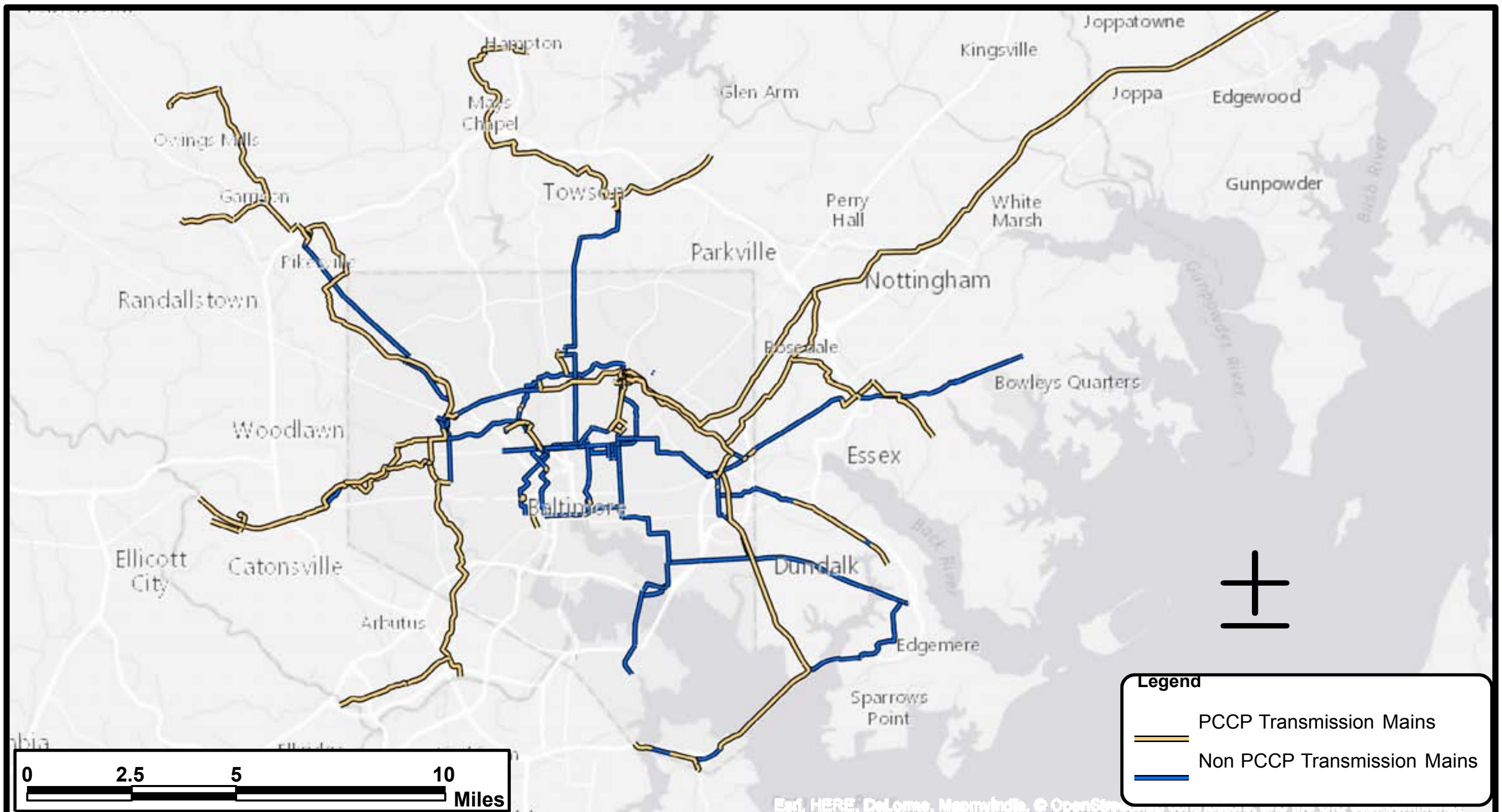

Figure 2. BMWD Large Diameter Water Mains

INSPECTION AREAS

In order to begin creating an inspection schedule, an inspection approach was needed. BMWD decided that all PCCP mains (within operational constraints) would be inspected acoustically for leaks, electromagnetically for broken wire wraps, and visually for any cracks or hollow sounding concrete. Additionally, BMWD thought it prudent to plan for the installation of Acoustic Fiber Optic (AFO) monitoring of the transmission mains. A single AFO Data Acquisition Unit (DAQ) can monitor up to 12 miles of transmission main. Therefore, the PCCP system was divided up into “DAQ Zones” which were lengths of pipes that could be monitored from a single location. Each “DAQ Zone” was centered on a BMWD-owned location which could house the DAQ computer system. These included but were not limited to pumping stations and water filtration plants throughout the city and county. Figure 3 is a map of the PCCP system divided into “DAQ Zones”

The DAQ Zone approach provided a monetary advantage to the BMWD. By pooling the inspection of what are considered to be separate transmission mains into one mobilization, BMWD saves significantly on technology costs, as well as the costs of utility workers supporting the inspections.

GENERAL APPROACH

A risk-based prioritization model was developed that utilized the available pipe attributes, historical break records, environmental factors and proposed DAQ zones to develop the condition assessment plan. In order to develop the prioritization model and the subsequent condition assessment plan, the following steps were required:

1. Data collection and evaluation (i.e. collected data and cleaned data)
2. Developed likelihood of failure (LoF) and consequence of failure (CoF) factors and design pipe risk calculations (i.e. added weighting factors and any other manipulations to LoF or CoF).
3. Ran the model (i.e. computed risk at the asset level)
4. Prepared a DAQ zone map
5. Calculated a risk roll-up to DAQ zone (i.e. transpose aggregated asset level risk to the proposed DAQ zones)
6. Prepared inspection plan based on aggregated DAQ zones risk

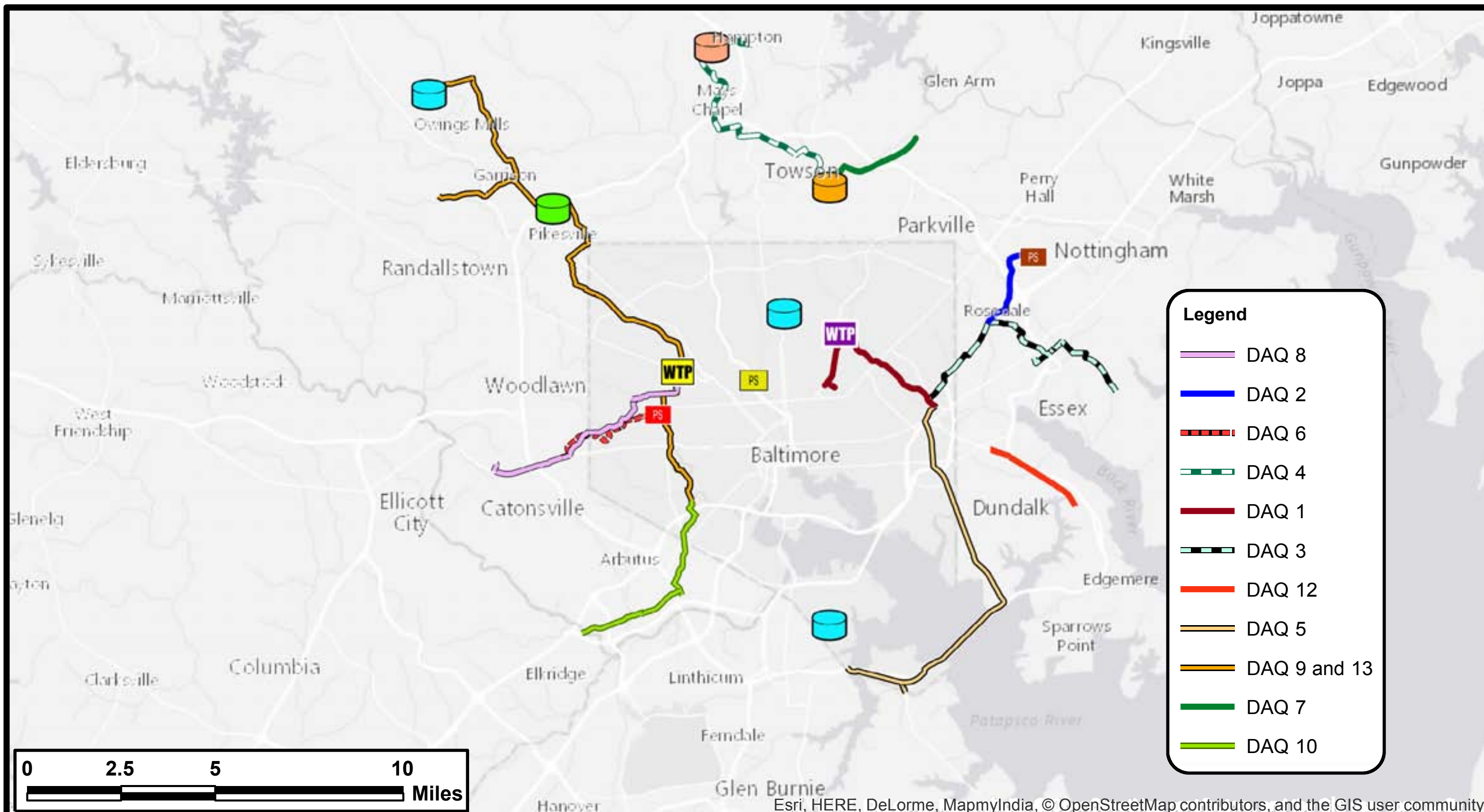

Figure 3 . DAQ Zone Map

AVAILABLE DATA EVALUATION

Much of the data required for the prioritization model had been collected and analyzed as part of the previous study. Additional data was also collected to further enhance the model. This data included:

1. Pipe Design Information
2. Pipe Age
3. Break History
4. Condition Assessment Data
5. Operational Data (Hydraulic Model)
6. GIS Data
7. As-Built Drawings

In order to overcome some of the limitations in the data, certain assumptions were made. These include:

1. For multiple designs within a single contract, the Risk model assumes the average likelihood of failure of all designs.
2. Data confidence scoring is based on data sources used
3. For pipe segments missing likelihood and consequence factors, the pipeline risk prioritization (PRP) module assigns likelihood of failure and consequence of failure factors using likelihood of failure and consequence of failure scores and the standard deviation of surrounding pipe segments within the same contract.
4. Pipeline installation dates are assumed to be accurate for modeling purposes.

LIKELIHOOD OF FAILURE FACTORS

In order to determine the likelihood of failure for the transmission mains the a structural analysis was conducted for all individual PCCP designs utilizing the design methods outline in the current AWWA C-304 Standard. The results of this analysis were incorporated as the Severity Rating and the Ps/Po factors. Severity Rating measures the state of stress or strain in the pipe under load from internal pressure and external soil and live load versus allowable limits. This rating indicates the limit state within which the pipe is operating. Therefore, risk will increase as severity ratings increase. Ps/Po is the ratio of working (service) pressure, Ps, to the zero compression pressure, Po, of the concrete core. Lower ratios indicate an increased pipe tolerance for surge and other unexpected loads without subjecting the core to tension and potential cracking. A list of all likelihood factors considered is shown in Table 1. The weighting for these factors along with the Consequence of Failure factor was determined during several workshops that included personnel from several different agencies and departments within the BMWD and their consultants.

Table 1. Likelihood of Failure Factor Weighting

LoF Factor	Weight (%)
Wire Class	16%
Cylinder Thickness	4%
Wire Diameter	4%
Severity Rating	6%
Ps/Po	4%
Failure History	6%
Leak History	4%
Data Confidence Score	4%
Installation Year	15%
Time Since Last Inspection	15%
No. of Wire Breaks	20%
Joint Anomaly	5%
Wire Break Zones	5%

$$\boldsymbol{Likelihood\ of\ Failure} = \sum_{i} \boldsymbol{Factor\ i\ Score\ \times Weighting\ of\ Factor\ i}$$

CONSEQUENCE OF FAILURE FACTORS

As with any risk-based prioritization, a measure of the consequence of a failure must be determined. The factors chosen are listed in Table 2 below.

Table 2. Consequence of Failure Factor Weighting

CoF Factor	Weight (%)
Waterways/Stream and Wetland	10%
Transportation/Urban Impact	15%
Railroad Impact	15%
Pipe Size	25%
Pressure	5%
Redundancy	15%
Critical Facility	5%
Large User	10%
Waterways/Stream and Wetland	10%
Transportation/Urban Impact	15%
Railroad Impact	15%
Pipe Size	25%
Pressure	5%

$$\boldsymbol{Consequence\ of\ Failure} \sum_{i} \boldsymbol{Factor\ i\ Score\ \times Weighting\ of\ Factor\ i}$$

MITIGATION STRATEGIES

Regarding risk mitigation factors, the primary intent lies within quantification of risk reduction. The following factors have been identified pertaining to BMWD pipes, each having a specific impact upon various LoFs and/or CoFs. Known LoFs affected include: 1) Time since last inspection; 2) Catalog remaining useful life (CRUL); 3) Failure history; 4) Leakage history; 5) Wire wrap break; 6) Wire wrap break zone; 7) Joint anomaly; 8) Percent to yield; 9) Material.

Known CoFs affected include: 1) Redundancy; 2) Pressure drop; 3) Large user; 4) Critical facility. Depending on mitigation strategy and available inspection data, additional likelihood and consequence factors will be incorporated as needed. Table 3 details these factors.

Table 3. Risk Mitigation Factors

Risk Mitigation Factor	Description
AFO	AFO Installation
Replacement	Affects All Pipes Replaced
Point Repair	Carbon Fiber, Pipe Stick, Tendon
Linear Repair	Slip-Lining, Structural Lining
System Strategies	New Redundant Pipe, Pressure Management
Carbon Fiber	Affects Percent To Yield
Valve Exercise	Future KPI

Total preliminary pipe risk is calculated for each pipe as the product of the likelihood of failure and consequence of failure factors:

$$Risk = Likelihood\ of\ Failure\ X\ Consequence\ of\ Failure$$

However, a risk mitigation factor (MF) is necessary as this factor imparts specific impacts upon both LoF and CoF factors. It is important to consider MF at this time to ensure all appropriate LoF and CoF factors are included. Otherwise, the risk calculation will not provide an accurate depiction of the current condition of a pipeline.

Risk will be mitigated by reducing individual contributing factors based on the mitigating action taken (i.e. inspecting a particular pipeline will alter the "time since last inspection factor" which will reduce risk). Upon consideration of risk mitigation factors, the total final pipe risk is now calculated for each pipe as the product of the likelihood of failure, consequence of failure and risk mitigation factor:

Risk = Likelihood of Failure (LoF Mitigation) X Consequence of Failure (CoF Mitigation)

With this in mind, the model provides computational risk at the asset level on a pipe section by pipe section basis. The computed risk on individual pipe assets can also be aggregated in various ways, such as per DAQ zone and pipe C.W.O. (Contract) number. Decision making can then be based on the average risk of the group of pipes. For this evaluation the proposed DAQ zone categorization was taken into account for the risk based prioritization. Accordingly, DAQ zone map was prepared, along with a calculation of risk rolled up to individual DAQ zones and an inspection plan based on risk as calculated for DAQ zones. Figure 4 presents the results of this analysis.

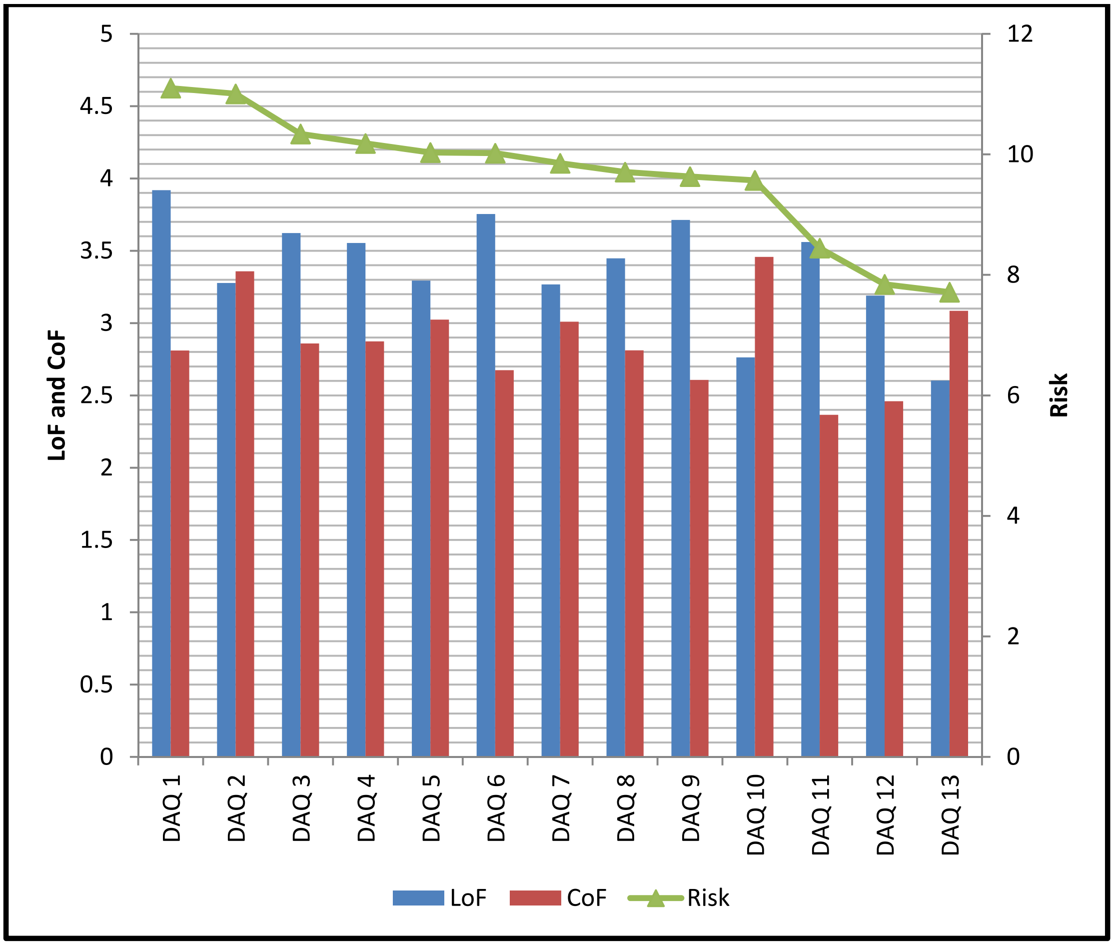

Figure 4. Results of PRP Analysis

CONDITION ASSESSMENT PLAN

Once the preliminary risk assessment model was developed, a system-wide condition assessment plan was prepared, including estimated budgetary needs to complete the work. The budget will dictate extent of inspection services provided each year. The goal is to address the pipeline of highest risk and, equally important, maximize BMWD's budget. In order to maximize the budget, pipelines may be inspected out of their risk order. As budget constraints are encountered, a pipeline projected for inspection may exceed current available funds within the existing fiscal year. Consequently, this pipeline will get pushed to the following year. At this point, the prioritization model will search through the list of residual pipelines requiring inspection until reaching the next highest risk pipeline that fits within the remaining available budget. It is anticipated that one or two DAQ Zones per year will be inspected based on these budget constraints. Table 4, below details the annual inspection plans.

Table 4. Condition Assessment Plan

Inspection Year	Length (Miles)	DAQ Zone
1	5.3	DAQ 1
2	2.6	DAQ 2
2	8.1	DAQ 3
3	8.3	DAQ 4
4	10.5	DAQ 5
5	3.9	DAQ 6
6	3.0	DAQ 7
5	6.5	DAQ 8
7	8.6	DAQ 9
Existing AFO System	5.9	DAQ 10
6	7.2	DAQ 11
Short Sections: To Be Inspected Separately From DAQ zones	2.9	DAQ 12
8	11.4	DAQ 13

Incorporation of PCCP Inspection and Monitoring Data

Upon completion of inspection, pipeline information will be uploaded and displayed on a web based data viewer for BMWD. The data viewer will contain both static and dynamic pipeline information and therefore not require AFO installation of every BMWD pipeline. This website will incorporate all available data on every inspected main in one location. This will include EM data, AFO wire break, structural analysis, and pipe specifications. This website will allow BMWD to view real-time condition of its most critical assets where AFO is installed, while remaining assets retain most recently collected condition data. The data and analysis provided by the website will be included in the risk assessment model contained in the pipeline risk prioritization module to allow planning for future rehabilitation or replacement. Figure 5 represents the data available on the website.

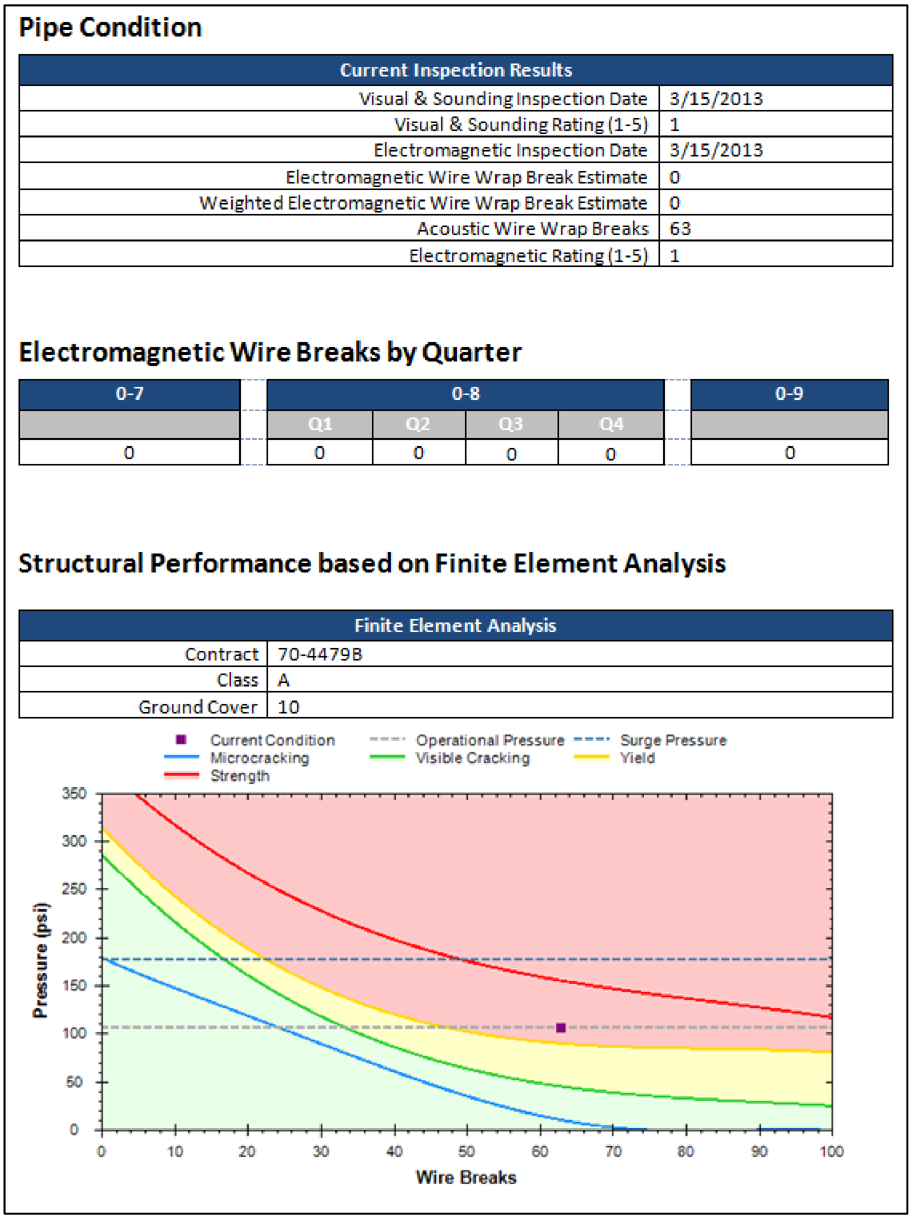

Figure 5. Example of Web Based Data

CONCLUSION

BMWD has embarked upon an ambitious 10 year plan to inspect and manage its large transmission mains. Through the use of risk-based prioritization and forward thinking, the plan has been optimized to reduce shutdowns, inspect the worst sections first and conserve budget. Implementing a comprehensive inspection program in conjunction with the monitoring and reporting systems, promotes timely repair of deteriorated pipe sections, optimizes the useful life of a pipeline, and provides condition assessment for asset management while maintaining a safe and reliable PCCP transmission system.

Driving the Industry Forward Again: WSSC's Pipeline Management System

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Abstract

The Washington Suburban Sanitary Commission (WSSC) is among largest water and wastewater utility in the United States serving over 460,000 customer accounts including approximately 1.8 million residents in Montgomery and Prince George's County Maryland (suburban Washington D.C.). Of the nearly 5,600 miles of water mains, approximately 145 miles are comprised of large diameter (greater than 36-inches in diameter) Prestressed Concrete Cylinder Pipe (PCCP). These PCCP mains make up the backbone of WSSC's large diameter water transmission system therefore placing a high consequence of failure for these pipelines. For more than five years, WSSC has successfully implemented a PCCP condition assessment program using state-of-the-art inspection, condition assessment, and monitoring techniques. Since its inception, the Program has evolved from a conventional PCCP condition assessment and engineering approach to an integrated, near-real-time pipeline monitoring and management system. This system has been built upon an online GIS based management system of the large inspection data sets (including live monitoring data) and engineering analysis allowing WSSC to manage their transmission mains in a dynamic manner. WSSC's data management system also goes beyond the node to node (valves, appurtenances, pressure zones, etc.) resolution and drives down to a pipe-by-pipe basis. Pipe by pipe resolution of the data allows detailed and directed analysis, statistical modeling, prediction, and planning where the individual pipe segments form the basis of the statistical samples. Automated alerts and up-to-date reporting have been incorporated into the online GIS based management system providing almost instantaneous access to all its pipeline information. This paper will provide an update to WSSC's PCCP management program including its recent evolution into an online GIS based information approach.

PROGRAM BACKGROUND

PCCP has been manufactured since 1943. WSSC's first PCCP pipeline was installed in 1945 and remains in service today crossing from the Patuxent Water Filtration Plant, behind WSSC's headquarters and crossing under I-95 heading toward Prince George's County. Since the 1950's, a total of 145 miles 36-inch diameter and larger PCCP were installed in the WSSC service area. Following a series of failures on large diameter PCCP Pipelines in the 1970's, WSSC began a PCCP inspection schedule which has eventually evolved into the successful program it is today. The first PCCP internal inspection was performed in 1981 by WSSC and Openaka (acquired by Pure Technologies in 2005). Over the following decades, WSSC continued to research, investigate and develop technologies to assess the condition of the different components in PCCP pipelines.

WSSC's Program consists of a five (5) to seven (7) year inspection and condition assessment cycle for each transmission main, with adjustments to the subsequent inspection schedules based on results of the condition assessments and long-term acoustical monitoring. The results of the inspection efforts are utilized to determine the current condition of individual pipe sections, which then are used to provide engineering recommendations for the rehabilitation, repair, or replacement of distressed PCCP pipe sections. Following these inspection and repair efforts, and prior to placing the pipeline back in service, an acoustic fiber optic monitoring system is installed in each inspected pipeline to track any future breaks in prestressing wires while the pipeline is in service (which complements the baseline inspection data) and provides a warning that a pipe is approaching the limits of its strength capabilities.

Each internal inspection methodology used in the Program provides a distinctly different data set used in the condition assessment of WSSC's PCCP pipelines. The approach of the Program is to implement multiple inspection techniques in order to compensate for the limitations of the individual technologies, as there is no one 'silver bullet' for PCCP assessment. This allows WSSC to make better rehabilitation, repair, or replacement decisions to best manage the PCCP pipeline network. By utilizing the various inspection and analysis methods presented below, as well as long term acoustic monitoring, WSSC has implemented one of the most comprehensive PCCP pipeline management programs in the world. The Program utilizes the following inspection, assessment, and monitoring techniques:

- Leak detection services using the SmartBall® or Sahara® acoustic technologies
- Above ground GPS survey of appurtenances
- Internal visual and sounding inspection
- Internal Electromagnetic inspection using PureEM technology
- Internal Sonic/ultrasonic inspection
- Internal inertial mapping of each transmission main
- 3D non-linear finite element analysis

- Hydraulic pressure transient monitoring
- Repair/rehabilitation oversight and inspection
- Long-term acoustic monitoring using the Acoustic Fiber Optic (AFO) technology
- Web/GIS based data management and reporting

UPDATE TO PROGRAM RESULTS

To date, WSSC has performed condition assessment and acoustic monitoring of over 115 and 85 miles of PCCP transmission mains 36-inches and larger, respectively. Figure 1 shows the inspected miles of PCCP by year since 2007.

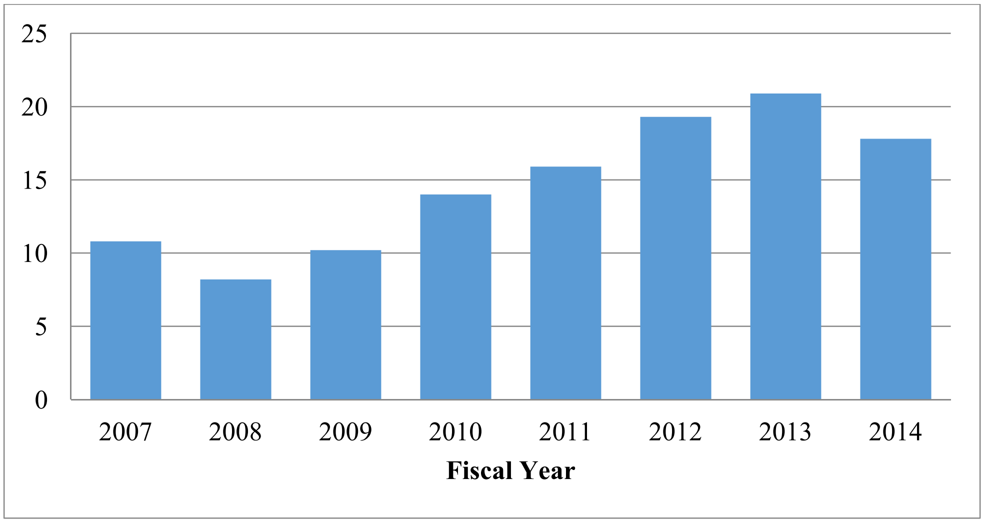

Figure 1: Miles of PCCP Inspected

Figure 2 provides a summary of electromagnetic results for WSSC's PCCP transmission mains through winter of 2014.

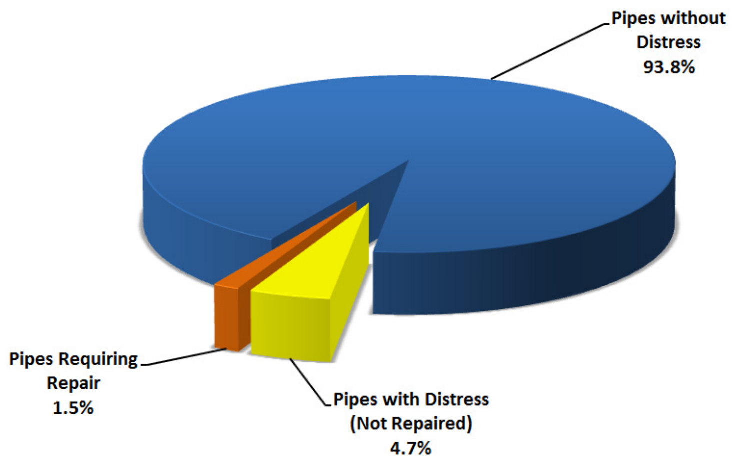

Figure 2: Summary of WSSC Inspection Results

The relatively low percentage of distressed PCCP found with WSSC transmission mains is consistent with hundreds of miles of inspection data collected for utilities across North America.

CONTINUOUS IMPROVEMENT

Robotic & Unmanned Inspections

As WSSC extends its advanced condition assessment program into the 36 inch and 42 inch PCCP, operational challenges will likely be encountered that were not previously seen in the larger PCCP. Primarily, access ports are likely inadequate for safe manned entry and traversal of inspection crews. Dewatering and ventilation problems in 36 inch & 42 inch mains are also new challenges found by WSSC's operational staff. Therefore, a robotic platform has been developed specifically for these pipelines. This tethered tool is capable of performing multi-sensor inspections in dry pipe or while submerged, with a range of up to 3 miles from a single access point. The benefit of conducting an inspection using the robotic platform is the ability to collect high definition visual and electromagnetic inspections rapidly without dewatering the pipeline or the need for manned entry. However, for most robotic inspections the pipeline will need to be removed from service (but not dewatered) so that the robotic device can be inserted into the pipeline. An Inertial Navigation System (INS) is incorporated into the robotic platform to assist in mapping the pipelines, thus allowing WSSC to obtain accurate location and alignment of the pipelines on a pipe-by-pipe resolution. To inspect in-service pipelines with electromagnetics, the PipeDiver™ deployment platform is utilized. PipeDiver is a device consisting of a battery module, electromagnetic, and a tracking module. PipeDiver can be inserted to a live pipeline via a simple hot tap connection and insertion sleeve, an existing access, or a submerged tank. Once inside the line, PipeDiver will travel with the water flow until it reaches a predetermined extraction point. PipeDiver movement and distance traveled (progress through the pipeline) is tracked from above ground via check points, similar to SmartBall. The system has been engineered to overcome the challenge of insertion and retrieval from a live pressured main. In May 2013, WSSC deployed the PipeDiver technology to inspect approximately 3 miles of the 54-Inch Prince George's County High Zone 1 Main. Recommendations based on the electromagnetic inspection resulted in 20 pipe repairs and 17 pipe replacements. A total of 11 pipes were identified as being highly distressed and of the 11 replacements, all were confirmed and validated destructively (continuity testing followed by external pipe dissection). Validation of all damaged pipes confirmed the results of the electromagnetic inspection avoiding a possible PCCP failure. Validation of the results of any inspection should be a fundamental part of any PCCP condition assessment program.

Inertial Mapping

Since piloting the technology in 2010, WSSC has fully incorporated inertial mapping into the majority of the inspections to improve the accuracy of the pipeline alignment and overall pipeline feature location. The inertial mapping can be deployed during manned inspections or via the tethered robotic platform. The inertial mapping information is also complimented by an appurtenance GPS survey and above ground survey performed on all inspections prior to deploying any internal inspection technology. This data is important when excavating problematic pipe for repair. Determining exactly where to excavate a pipe section for repair is not a simple task when a pipe section is several hundred or thousands of feet from the nearest known feature (e.g. air valve). Having GPS coordinates for every PCCP pipe joint on the pipeline significantly increases the confidence of repairing the correct pipe section following inspection or when the AFO system indicates an emergency repair is necessary. Excavations and validations performed after locating individual pipe segments have shown accuracies of up to 1.5 meters from the reported GPS location. In September 2013, WSSC performed a right-of-way (ROW) walkover and above ground GPS survey of the Prince George's County High Zone 1 "PGHZ1" 54/42/36-inch Transmission Main. The purpose of the ROW walkover and GPS survey of PGHZ1 was to locate sections of exposed pipe in streams and storm water drainages, collect coordinates of appurtenances, and assess the ROW with respect to buildings, structures and general landscape undergrowth cover. Prior to this survey, WSSC had already discovered several sections of exposed PCCP pipe. As a result of the walkover, a 30-foot high berm was identified on top of the pipeline ROW suggesting additional earth loading had been placed over PCCP. Other findings of the walkover included missing manhole covers, heavily forested areas, ROW encroachments and exposed PCCP at various locations. Incorporating the ROW and GPS survey results with the AFO system allowed WSSC to pinpoint the locations of all individual pipes with distress.

AFO Emergency Response System Development

Typically, wire break identification and location analysis takes place during normal business hours. However, should a series of wire breaks occur on a pipe outside of these hours, the chances of them being identified prior to analysts arriving the next working day were low. Driven by the need for a more rapid wire break identification method, WSSC and Pure staff developed real time analytical techniques that provide wire break identification along a single transmission main based on user-defined parameters. Initial developments on the automatic alert system notified users via email if three or more wire breaks have occurred on a specific transmission main over a 24-hour period. The updated alarm system can be customized to send notifications based on a range of risk based parameters including, frequency and/or quantity of wire break events, surge pressure data, structural analysis and others. This process has been in place for WSSC's transmission main system since early 2011 and has notified WSSC of several rapidly deteriorating pipes prior to a failure. The alert system can also identify all valves and/or other appurtenances required to isolate a specific distressed pipe section. In November 2014, the AFO system detected significant and accelerating prestressing wire break activity on a single pipe on a 66-inch PCCP main. The total number of wire breaks observed approached, and then exceeded, the Yield Limit for the pipe, which was determined through structural modeling. This pipe featured an exceptionally unique configuration — the pipe was half-contained inside a 108-inch extended steel tunnel casing and is near a tee, reducer, and two inline gate valves. Less than 25 feet from the pipe of concern is an active industrial and commuter railroad. An inoperable 54-inch in-line valve scheduled for replacement in spring of 2015 impacted the shutdown of the main for rehabilitation. The high likelihood of failure and significant consequence of failure exposed WSSC to a high risk. Thus, after continuously monitoring the pipeline and careful deliberations it was decided that the pipeline should be shut down and dewatered in order to repair the pipe of concern under controlled emergency conditions. The AFO Emergency Response System allowed WSSC to safely make the necessary arrangements logistically and operationally to shut down the pipeline in a rapid manner, repair the distressed pipe and return the main to service in a short period of time with minimal impact to the water system.

PureGIS & Web-Based Pipeline Management Systems

The GIS based management system is a customized web-based service for the storage, retrieval, and display of gathered inspection data to augment WSSC's review and use of the Program's inspection results. The web-based service is used to display the inspection and real-time monitoring data for WSSC's PCCP Program. The web-based service allows pipeline information to be exported in ESRI compatible GIS shapefile, excel data table, or individual pipe segment detail word files. Attributed data within the GIS application includes individual pipe specification information, inspection results (current and historical), structural modeling information, repair history, wire break trend analysis, dynamic risk analysis, pressure data, as well as several other customizable features. The system also supports a document management function to upload digital pictures, drawings and other digital media to an individual pipe section, node to node, or the transmission main as a whole. The web-based service is accessible via any mobile device with internet access and can be accessed at virtually any time. All the information pertinent to a pipe section is readily available and displayed in a format that was designed specifically to this Program. Because the data is hosted in a database, WSSC has the ability to query the information contained in the database to make decisions in an efficient and effective manner. The web-based system is also setup to provide overall system status and condition of the pipelines or group of pipelines in WSSC's system. Figure 3 shows WSSC's GIS based management system. The GIS based management system incorporates near-real time data with the structural and advanced statistical analysis models to produce dynamic reporting that helps WSSC manage their PCCP mains successfully. By combining the structural analysis with condition data, estimates of when the pipeline should next be inspected can be completed. To do this, Pure has developed a statistical simulation that utilizes failure history, inspection data, structural analysis, and permanent monitoring data for PCCP. For the re-inspection model, static data (failure history, inspection data, etc.) and historical acoustic monitoring data can be used to project deterioration over a given number of years. Using this model, WSSC can then prioritize re-inspections of individual contracts or full transmission mains thereby managing risk in a dynamic manner. An example analysis performed for WSSC on the River Road and Rock Creek transmission mains. The analysis focused on the likelihood of failure for individual pipes that were separated into contracts for both transmission mains. Contracts are bound by inline valves thereby allowing for a more convenient inspection approach. The analysis used baseline condition and ongoing AFO monitoring data to prioritize future inspection for the pipelines by contract lengths. The analysis showed that the likelihood of failure would focus the next inspection towards the Rock Creek pipeline while the consequence of failure weighed the next inspection towards River Road due to the geographic location of the main.

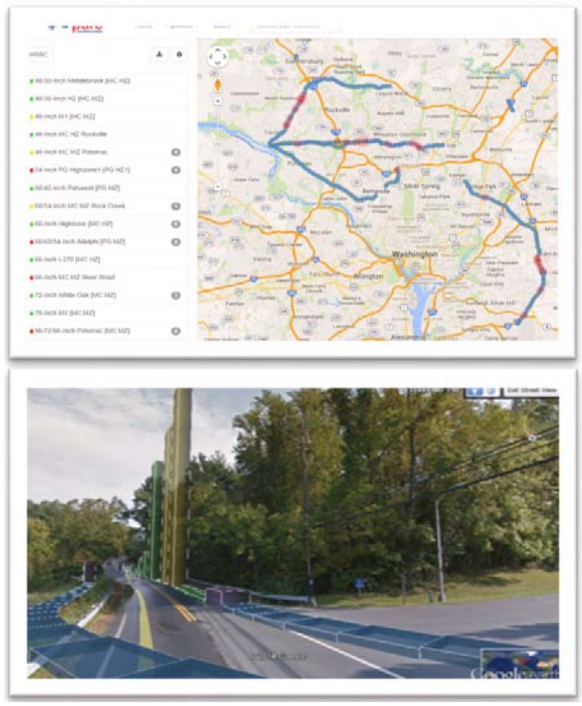

Figure 3: GIS based management system

CONCLUSIONS

WSSC's PCCP Management Program continues to lead the industry in condition assessment of PCCP mains with its state-of-the art inspection, acoustic monitoring and web-based management tools. The PCCP Management Program continues to be the safest and cost effective approach to identify, manage, mitigate and reduce the risk of PCCP failures (3).

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Asset Management Mixing Bowl: Idea Sharing Amongst Owners

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Abstract

Many municipalities have the same concerns when it comes to infrastructure planning. Whether it is condition assessment, budgeting or general planning concerns, most similar sized agencies face the same dilemmas throughout the U.S. The knowledge gained through active communication between those sharing a similar interest can be vast. It is likely that challenges being faced by one owner are also being experienced by many more similarly sized agencies. Howard County Department of Public Works (DWP) recently completed a condition assessment for over 44,000 LF of distribution main in one of its oldest planned communities. The Wilde Lake area, which was established in the mid-1960's, has experienced numerous water main breaks in recent years. In an effort to remain pro-active, Howard County DPW launched a comprehensive study into the cause of the water main breaks, with the intent of developing an overall replacement strategy for the community. As the project developed, it became apparent that this community's distribution system was a good representation of the County's system as a whole and the studies completed as part of this project could be applied comprehensively to the entire system. As such, the Wilde Lake condition assessment became a pilot program which could be used to develop a larger asset management program for their distribution system. As the project continued, it became apparent that other local agencies were facing a similar dilemma of how to evaluate and manage their distribution systems. In an effort to gain an industry-wide perspective, the County developed a "Pipeline Management Working Group" that included representatives from Baltimore County, Baltimore City, DC Water and WSSC. These agencies met both in person and via webinar to discuss topics such as:

- Various inspection techniques
- Desktop pipeline risk analysis
- Data Management
- Replacement strategies
- Operational strategies

Following the successful outcome of the local information sharing session, the program was expanded to include other North American utility owners, when Howard County hosted a Pipeline Management Working Group at the 2014 ASCE Pipelines Conference in Portland, OR. This session was attended by owners from the US and Canada, all of whom shared a common interest in learning how each other handled their distribution systems.

The idea of information sharing, although not a new concept, it is typically done only on a local level. However, by expanding the circle of participants to those outside a local region, additional perspectives can be gained. As the mixing bowl continues to grow to include additional participants, the level of quality knowledge being exchanged is sure to reach new heights!

OVERVIEW OF HOWARD COUNTY AND ITS WATER SYSTEM

Howard County is located in the central part of Maryland and borders six surrounding counties as shown in Figure 1. Howard County purchases the majority of its potable water from the City of Baltimore which is conveyed through a series of transmission mains through Baltimore into Howard County via three (3) large master meter connections into the County.

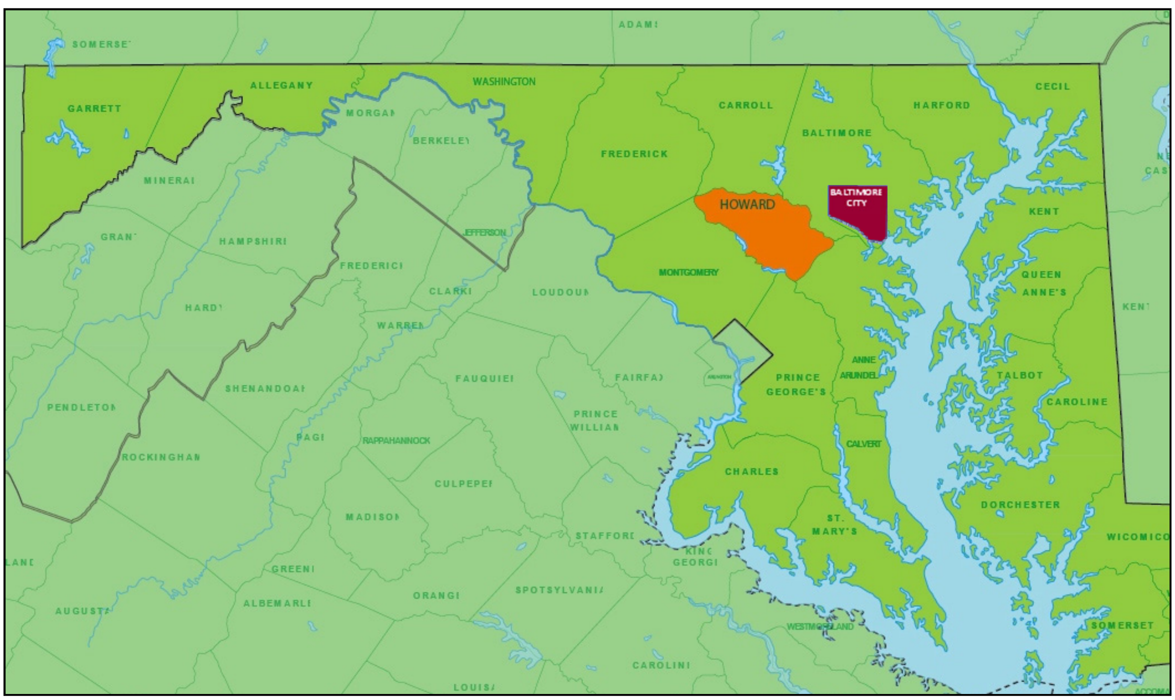

Figure 1: Location of Howard County and Baltimore City, Maryland

Howard County's water system consists of:

- More than 1,000 miles of water main
- Approximately 900 miles of Cast Iron / Ductile Iron Pipe (DIP)
- Approximately 100 miles of PCCP or Plastic Pipe

- Most transmission mains are PCCP and Ductile Iron
- Average Daily Requirement of 26 MGD
- 10 MG of water storage

WILDE LAKE WATERMAIN CONDITION ASSESSMENT

The older parts of Howard County were developed in the mid 1960's; as such the water distribution system consists mostly of DIP that was installed in the 1960's and 1970's. Figure 2 shows a map of distribution mains in the Wilde Lake neighborhood. The County's distribution system is aging and has increasingly been experiencing pipeline breaks in recent years. Although a comprehensive program for monitoring and evaluating the condition of the larger PCCP transmission mains has been well established in recent years, the County, similar to other municipalities of similar size, had not implemented a program for management of their distribution sized mains (less than 16-inch diameter).

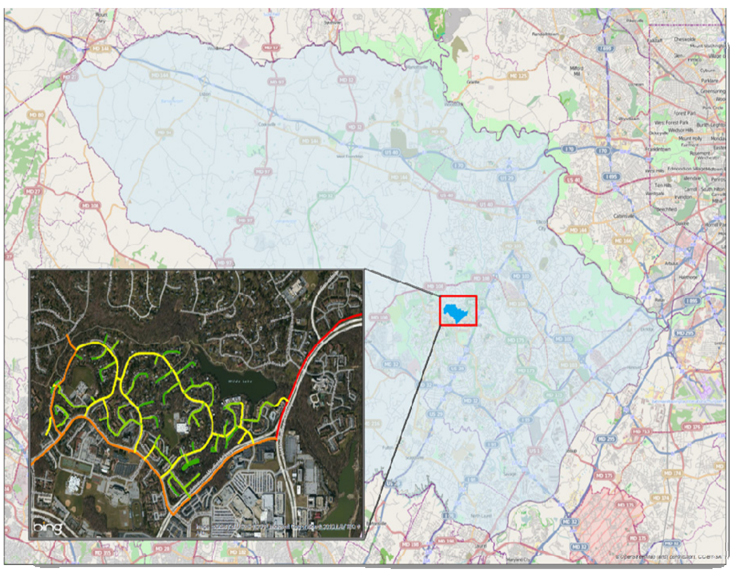

Figure 2: Overview map of distribution mains in Wilde Lake neighborhood

The Wilde Lake community in Howard County experienced approximately thirty-five breaks in a fifteen year period. A full corrosion evaluation was performed in an effort to evaluate the cause of the breaks. The results of the evaluation indicated that the ductile iron mains in the area may be subject to damage caused by corrosion from stray currents emanating from an impressed current system used to protect gas mains, as well as from low resistivity soils.

In an effort to maximize the life of the water mains in the Wilde Lake area, working with Pure Engineering Services (PES), the County completed an evaluation to determine the extent and magnitude of the damage to the existing water mains in the Wilde Lake area. The evaluation (as depicted in Figure 3) included:

- Data Collection & Evaluation (Desktop Situation Analysis)
- Preliminary Risk Assessment
- Condition Assessment Planning

Ultimately, this evaluation resulted in recommendations for extending the service life of the mains that were still in good condition, as well as to develop a plan to systematically replace the mains that were found to be at high risk for failure.

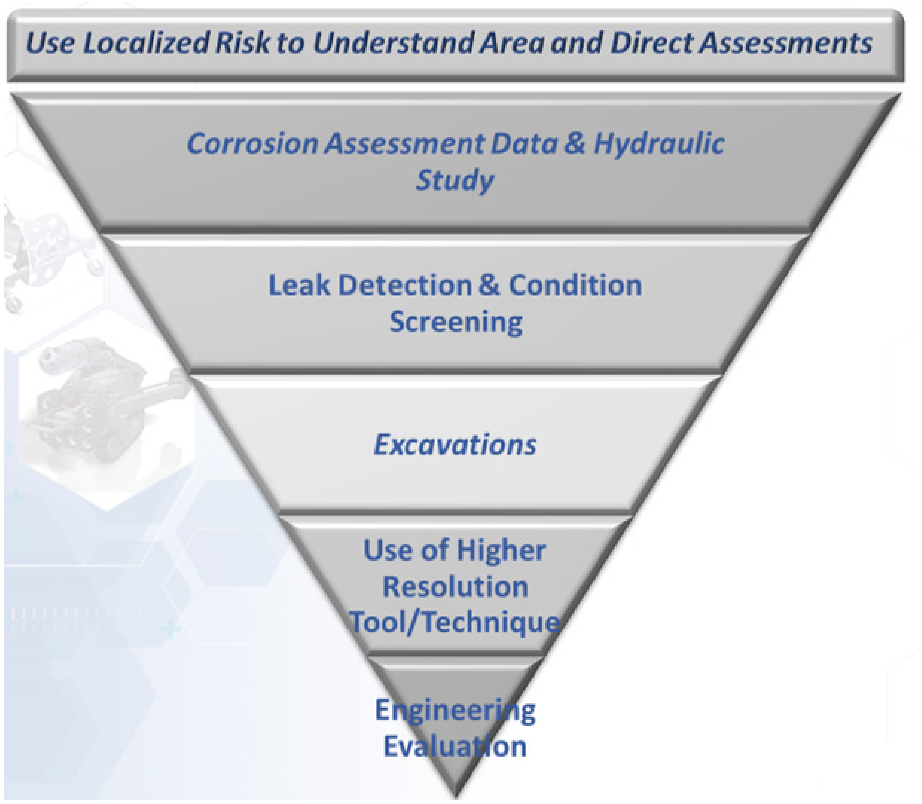

Figure 3: Approach used for technology deployment for the Village of Wilde Lake

The County quickly realized that the Wilde Lake area was a good representation of the County's distribution system as a whole and that the results of the evaluation on the water mains in Wilde Lake could be applicable to their entire distribution system. As such, the County, working with PES, developed a comprehensive strategy for asset management by using the Wilde Lake area as a pilot system.

PIPELINE WORKING GROUP – ROUND 1

As the Wilde Lake pilot project progressed, it became apparent that the challenges encountered in Howard County were likely the same challenges other similarly sized municipalities were facing. In an effort to exploit the knowledge of a larger group of like-minded owners so that each could benefit from the others, the County organized a working group of local municipalities in the Baltimore / Washington, DC area (Figure 4). The intent of the working group was to allow for the free flow of ideas, thereby opening up pertinent discussions on the challenges of planning and implementing an asset management program for water distribution mains.

Figure 4: Screenshot of materials for first Working Group meeting

The first working group included representatives from Howard County, the City of Baltimore, the Washington Suburban Sanitary Commission (WSSC) and DC Water. Although the focus of the working group was intended to be on pipeline management, the discussion also incorporated other challenges the owners had been recently facing. Major topics of discussion included:

- ***Inspection techniques*** – each had tried different techniques that yielded varying results. The overall consensus was that data collection techniques were improving, allowing for a more informed decision making process.

- ***Data management*** – the means of data management varied between agencies and within each agency depending on the information being sought. Various software platforms and data management techniques had been successfully employed by each of the owners.

- ***Pipeline materials*** – ductile iron pipe (DIP) was the most commonly used material for distribution main replacement, although polyvinyl chloride (PVC) was also used where applicable (highly corrosive areas).

- ***Means for Corrosion Protection*** – methods of protecting the metallic pipelines were discussed, which ranged from polyethylene bags to sacrificial anodes.

- ***Operational Strategies*** – approaches to programs such as pipeline flushing and valve exercising were discussed in detail.

Feedback from the initial working group was positive. It quickly became evident that the collaboration between owners was a beneficial process. By facilitating an open forum discussion, the working group environment allowed for the exchange of ideas between individuals who would normally not have the opportunity to share this information. A second working group was planned – but this time on a much larger scale.

PIPELINE WORKING GROUP – ROUND 2

Based on the success of the local working group, the County arranged for a pipeline management working session during the 2014 ASCE Pipelines conference in Portland, OR. The idea behind this session was to open the discussion up to a larger audience and gain input from outside of the Baltimore / Washington, DC metropolitan area. By hosting the working group at this conference, the attendees already had a common interest, allowing for a high probability that they too faced similar challenges.

The continuing goal of this Working Group was to gain an industry wide perspective as well as develop and share successful strategies on the management of water distribution systems. This second working group hosted by Howard County included attendees from Alaska, Georgia, Texas, Colorado, Arizona, and Ontario. This cross-section of utility owners from all over North America provided a unique cross-section of utility owners, each providing input beneficial to the others.

The major topics of discussion for the working group included:

- What has worked / What has not worked in regards to pipeline asset management

- Pipeline Assessment Programs

- Technologies – Inspection, Analysis, Renewal

- Methods of Data Management

- Successes and Failures

One of the topics with the highest interest was a discussion on what each owner had done that had worked and what not worked with regards to asset management and pipeline renewal strategies of small diameter systems. It became apparent that similar to the discussions on the local level, most utilities were using desktop models for small diameter replacement strategies, which were being augmented with active leak and break data.

Another topic that was discussed in detail was on the use of different technologies and condition assessment data. Some of the replacement strategy prioritization approaches discussed included:

- The use of break data
- Data gained from pipe coupons
- Forensic analysis of failed pipe
- Leak detection
- Valve and hydrant assessment and maintenance programs
- Consequence of failure (Example shown in Figure 5)
- Contingency plans

The second working group, held on the national level, allowed for owners with a common interest to interact in a workshop-type setting and gain valuable information from one another with regards to managing their water distribution systems. This interaction between utilities proved to be a helpful mechanism to gain an industry wide perspective on the management of distribution systems.

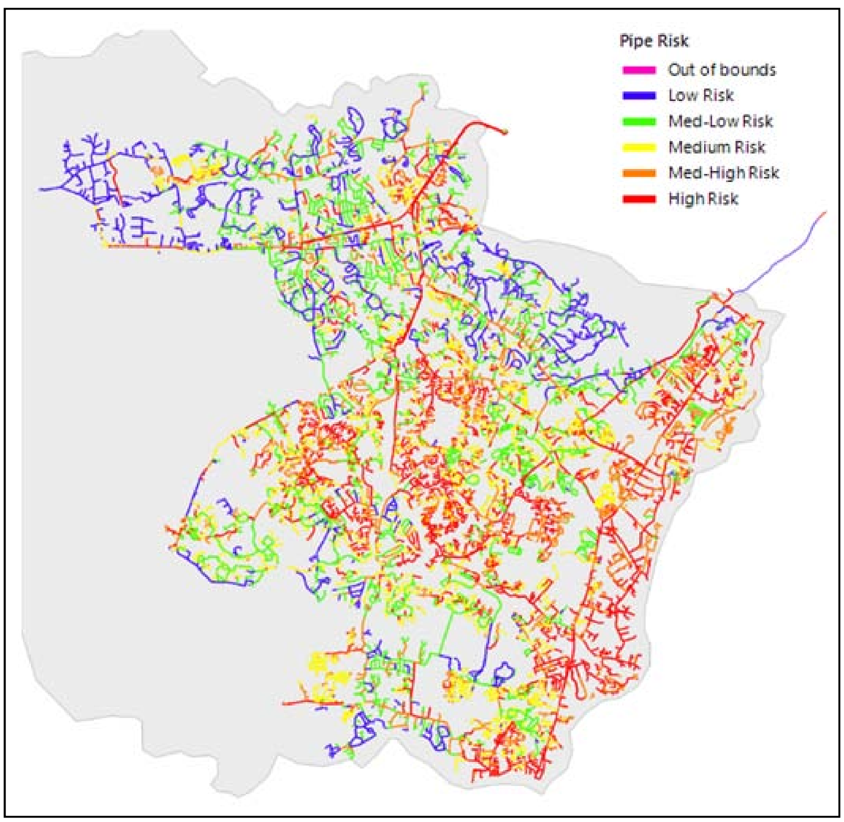

Figure 5: Consequence of failure map for the Village of Wilde Lake

PIPELINE WORKING GROUP – MOVING FORWARD

In an effort to increase the keep the process of sharing information moving forward, a third round of the working group is scheduled for April, 2015. This session is planned to be at the local level in the Baltimore / Washington, DC area once again, with the addition of other similarly sized owners, who will join the conversation via an interactive video teleconference. The intent is to invite owners of similar size who are in the process of implementing pipeline management programs similar to what Howard County is currently doing to share the results of the pilot program and further discuss what is working, what is not working, as well as future initiatives.

Following this third working group session, a fourth session is also planned for the 2015 ASCE Pipelines Conference, which happens to be held locally in the City of Baltimore! This working session will allow Howard County to showcase their pro-active approach to distribution system management to their peers both locally and nationally.

CONCLUSION

Utility owners throughout the United States are facing significant challenges in managing their buried water infrastructure. These challenges have been documented by various industry reports placing a price tag on the buried pipeline infrastructure renewal needs in the hundreds of billions, if not trillions of dollars. In an effort to remain pro-active and remain at the forefront of the industry for innovative and responsible approaches to pipeline management, Howard County completed a condition study to evaluate the integrity of their public water system in order to provide fire protection and maintain domestic water service.

Interaction between utilities is one of the best mechanisms for information sharing, troubleshooting, and simply finding out what works and what doesn't. Understanding that other owners were facing a similar task, Howard County developed a Pipeline Management Working Group with the intent of bringing like-minded owners together to discuss this topic. This allowed for everyone to gain an industry wide perspective on maintaining and managing a water distribution system. The information shared during these working sessions was invaluable to the owners, each gaining the information that they "want to know" from others facing the same challenges.

Smart Pipeline Infrastructure Network for Energy and Water (SPINE)

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Abstract
The United States is critically dependent upon more than 5 million miles of pipeline (lifeline) infrastructure systems to transport water, natural gas, oil, and nuclear facility coolant and waste. Pipelines provide the lifeblood to society by transporting energy, water, waste, and other critical services; yet, the pipeline infrastructure in North America is inadequately prepared to support a growing economy dependent on sustainable growth, public health, and community resilience. Approaching pipeline systems installation, operation, and retrofitting by continuing to use the same 20th century processes, practices, technologies, and materials will likely yield the same results: increasing instances of service disruptions, higher operating and repair costs, and the possibility of catastrophic, cascading failures. How a nation operates, retrofits, and expands its pipeline infrastructure will help determine the quality of life for future generations and that nation's competitiveness in the global economy. If a nation is to meet important challenges of the 21st century, a new paradigm for the building and retrofitting of critical pipeline infrastructure system is required, one that addresses the conflicting goals of diverse economic, environment, societal, and policy interests. This paper presents the workshop outcome focused on transforming the nation's capability to plan, design, install, monitor, control, retrofit, and asset manage energy and water pipeline infrastructure systems to be both resilient and sustainable.

INTRODUCTION
Four years ago, America's energy infrastructure system earned a "D+" and the water infrastructure system earned a "D" on its report card, issued by the American Society of Civil Engineers (ASCE). Unfortunately, not much has changed. The professional society gave energy and water infrastructure a D+ for 2013. Pipelines crisscross our communities near our homes and schools, yet little attention is paid to this critical infrastructure until catastrophic failures occur. Virginia Tech and partner Universities (Carnegie Mellon University, Georgia Tech University, Louisiana Tech University, and University of Puerto Rico – Mayaguez) hosted a workshop titled "Smart Pipeline Infrastructure Network for Energy and Water (SPINE)" at the Virginia Tech Center in

Alexandria, VA. Attendees included industry leaders from the water, chemical, nuclear, hazardous materials, oil, and gas fields. Government and organizational officials from the U.S. Department of Homeland Security (DHS), the U.S. Congressional Research Services (CRS), the U.S. Environmental Protection Agency (EPA), the National Institute of Standards and Technology (NIST), the U.S. Department of Energy (DOE), the U.S. Department of Transportation (DOT), the National Transportation Safety Board (NTSB), Electric Power Research Institute (EPRI), Water Environment Research Foundation (WERF), and Gas Technology Institute (GTI) also participated in this SPINE workshop. This workshop was designed to sharpen the SPINE vision and mission, both of which are focused on transforming the energy and water pipeline industry to make it sustainable and resilient. A 1.5-day workshop jointly sponsored by Virginia Tech and partner universities was held at the Virginia Tech Center in Alexandria, VA. With invited researchers from academia, utilities, industry, and federal institutions the workshop identified opportunities and knowledge gaps relative to critical areas of sustainable and resilient pipeline infrastructure systems. The goal of the workshop was to develop a prioritization that can guide fundamental and applied research at federal institutions and entities funding research in energy and water pipeline infrastructure. Key question of the workshop is "how to create an intelligent, responsive continent-wide pipeline infrastructure that is fully monitored and dynamically controlled to allow for higher: reliability, cost-effectiveness, efficiency, sustainability, and resiliency."

PURPOSE OF THE SPINE WORKSHOP

Pipelines provide the lifeblood to society by transporting energy, water, waste, and other critical services; yet, the pipeline infrastructure in North America is inadequately prepared to support a growing economy dependent on sustainable growth, public health, and community resilience. The workshop started with several keynote presentations to frame the topics. The keynotes were followed by Thrust sessions to develop and rank potential research and educational activities. The workshop concluded with all the attendees developing a consensus on a prioritized list of research and educational goals. We have identified five major thrust areas (Figure 1): Performance & Durability, Sensing & Diagnostic, Installation & Retrofitting, Infrastructure & Society, and Education.

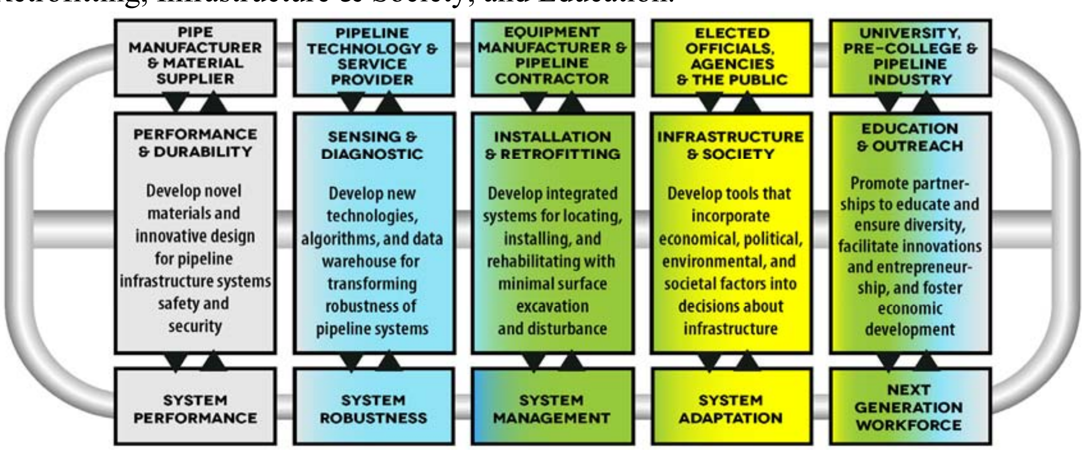

Figure 1. Research and Education Goals Integrated with Stakeholders and Outputs

10-year Vision: Advances in Knowledge, Technology and Education

Within the next 10 years, SPINE will establish itself as an agent for transforming the nation's capability to plan, design, install, monitor, control, retrofit, and asset manage energy and water pipeline infrastructure systems to be both resilient and sustainable.

Engineered Systems, SPINE will provide:

- Pipeline infrastructure systems that are fully monitored and dynamically controlled to allow for higher: reliability, cost effectiveness, efficiency, sustainability, security, and resiliency.

Enabling Technology, SPINE will provide:

- Novel materials and innovative designs that meet high-performance, resiliency, and sustainability requirements, and extend cost-effective service life;
- New technological solutions, such as robotics, wireless sensors, and energy-harvesting sensors, to improve robustness, particularly their safety and security;
- Advanced mathematical algorithms for real-time monitoring, diagnoses, and control of pipeline infrastructure systems for advanced asset management;
- Critical improvements in remote-controlled trenchless technologies to allow accurate operations in variety of sites and to monitor operational condition;
- Efficient and semiautonomous communication about updated site and pipeline systems data to the permanent pipeline data warehouse for risk management; and
- A dynamic platform for innovative decision-making that incorporates economic and social factors into decisions about sustainable and resilient pipeline systems.

Fundamental Knowledge, SPINE will provide advances in understanding:

- Failure modes and mechanisms to predict relevant pipeline infrastructure system performance criteria that promote resiliency and sustainability;
- Pipeline deterioration processes and methods of pipeline design and operation that meet necessary cost, installation, and life-cycle performance criteria;
- Capabilities and limitations of pipeline sensing and diagnostic techniques;
- Methods for accurately locating, inspecting, installing, and retrofitting pipelines;
- Data, information, and technologies for managing pipeline assets at scale; and
- Metrics for economic, policy, governance, environmental, and societal impacts.

Workforce Development, SPINE will:

- Educate and train engineers for careers related to pipeline infrastructure, with a mastery of cross-disciplinary knowledge through multidisciplinary teaming;
- Incorporate the "hands-on" training possibilities afforded by the pilot-scale and utility test- beds into education and outreach programs; and
- Increase the number of women, minority, and disabled entering the field.

Innovation Ecosystem, SPINE will:

- Establish partnership with pipeline industries and national, state, and local organizations to catalyze entrepreneurship and economic development;
- Form an alliance between pipeline stakeholders who currently confront fundamentally similar scientific problems, by leveraging Virginia Tech and Georgia Tech's NSF I-Corps; and
- Work closely with the Department of Economic Development to eliminate research barriers, establish industry and researcher-friendly intellectual property policies, engage seed and venture capitalists, and firmly commit to technology transfer and economic development.

SUMMARY OF KEYNOTE PRESENTATIONS

During the course of one and half days, the workshop provided academic and practical context and ideas for the workshop goals by keynote presentations.

The brief summary of each keynote presentation is presented below.

a) **Overview of the Smart Pipeline Infrastructure Network for Energy and Water (SPINE) Workshop:** Sunil Sinha, Professor and Director, Civil and Environmental Engineering, Virginia Tech, Blacksburg, Virginia.

Dr. Sinha presented SPINE's vision for smart pipeline systems that will (a) integrate planning, designing, installing, operating, and retrofitting, as well as social, economic, and environmental objectives for both legacy and new pipeline; (b) formulate and apply a multi-system, multi-hazard, and multi-criteria framework for pipeline assessment and management that spans considerations ranging from individual pipeline component performance to societal, economical, and environmental objectives; and (c) develop and apply innovative materials, technologies, and methodologies to enhance resiliency, sustainability, and security of pipeline systems. SPINE will provide the backbone for our nation's more than 5 million miles of energy and water pipeline systems through research and development of high-performance, fully monitored, resilient, and safe pipeline infrastructure, creating a new paradigm of transformative research to ensure the future of national energy security, economic prosperity, and quality of life. This massive effort will address not only a critical pipeline problem facing the nation, but will provide also an impetus for a resurgence of the U.S. industry involvement in pipeline infrastructure.

Salient Points: Establish National Center for Energy and Water Pipeline Infrastructure; Energy Pipeline (Oil, Gas, Nuclear, Hydrogen, Fracking, Carbon); Water Pipeline (Drinking Water, Wastewater, Storm water)***;*** High-performance, Fully-monitored, Resilient, and Safe Pipeline Systems; and Establish National Pipeline Education, Training, and Outreach Program.

b) **Keynote Speaker: National Institute of Standards and Technology (NIST), Pipeline Sustainability and Resiliency**: James Fekete, Group Leader, Applied Chemicals and Materials Division, Boulder, Colorado.

Dr. Fekete discussed the efforts by NIST on the condition assessment and life prediction of the pipeline infrastructure. He covered the need for service life models and the need for big data accumulation from various resources to evaluate the correlation for the future modeling efforts and decrease uncertainty. He briefly covers NIST's current database efforts (namely the damage and flaw database). He concluded his speech by discussing the future research needs such as the new methods for evaluating material properties, and thermodynamic properties of pipes.

Salient Points: Pipeline Condition Assessment and Life Prediction**;** Big Pipe Data (similar to human genome) need rules to evaluate uncertainty**;** Measurement Science – what we are measuring**;** Improve NDE – uncertainty in measurement**;** Real world artifacts – damage and flaw database**;** Thermodynamic properties research needed**;** Need service life models**;** Need methods for evaluating material properties**;** CO_2 pipelines - mitigate bio-corrosion**;** and H2 steel pipelines - multiple physics problem.

c) **Keynote Speaker: U.S. Environmental Protection Agency (EPA), Drinking Water and Wastewater Pipeline:** Dan Murray, Senior Environmental Engineer, Office of Research and Development, Cincinnati, Ohio.

Mr. Murray introduced the goals and approach of U.S. EPA's research program to evaluate and demonstrate innovative technologies in order to improve the cost effectiveness of the operation, maintenance, and replacement of aging and failing drinking water and wastewater infrastructure. Condition assessments, rehabilitation of wastewater and water distribution systems were briefly introduced, followed by advanced concepts of new innovative infrastructure designs including technologies for wastewater and reuse of water. He discussed the challenges in the determination of the water pipeline performance. He covered the recent U.S. EPA efforts on the subject matter such as the utilization of hydrants as rapid response system and the extension of the EPANET for advanced mathematical modeling and analysis.

Salient Points: Use Water Hydrant as rapid response system**;** Determining performance status is a challenge, improved water quality, and cost effective repair and replace program; New Purple Pipeline (two separate pipeline - Drinking and Raw Water)**;** and Extension of EPANET for water infrastructure analysis.

d) **Keynote Speaker: Electric Power Research Institute (EPRI), Nuclear Piping Infrastructure System:** Gregory Selby, Director, Nondestructive Testing and Evaluation, Charlotte, North Carolina.

Mr. Selby started his speech by acknowledging the current characteristics of the pipeline infrastructure systems. Various pipeline materials such as the steel, cast iron, concrete and PVC and the nature of these materials were discussed. Another aspect of the pipeline infrastructure discussed by Mr. Selby is the vast varieties of materials carried by pipelines (water, gasses, chemicals, oils, radioactive fluids). He mentioned that leaks in buried piping resulted in considerable political pressure and many piping systems are very difficult to access. Mr. Selby also discussed EPRI BPWORKS software to manage data related to buried pipe systems and perform risk ranking to prioritize the inspections of those systems subject to degradation. He concluded that future research is needed to assess better life predictions such as evaluating the corrosion based on different material types and possibly a matrix depicting different degradation mechanics of different materials.

Salient Points: A lot of buried pipe - steel, cast iron, ductile iron, concrete, PVC, etc.; Carries raw water, gases, chemicals, oils, and radioactive fluids**;** Extending nuclear piping life from 40 to 80 years**;** Development of Material degradation matrix**;** and Research to Commercialization and Application Initiative.

e) **Keynote Speaker: U.S. National Transportation Safety Board (NTSB), Pipeline Failure Analysis:** Ravindra Chhatre, National Transportation Safety Board, Pipeline Safety Division, Washington D.C.

In his speech, Mr. Chhatre described NTSB's organization, authority, and accident investigation process with an emphasis on pipeline accidents. He explained that NTSB does not regulate transportation equipment, personnel or operations and has no official role in establishing and enforcing industry regulation. NTSB is charged by Congress to investigate accidents in modes of transportation; assist victims of transportation accidents and their families; and has authority defined under U.S. Code Title 49, chapter 11. He discussed some recently completed major pipeline accident investigations. He concluded by discussing the future research needs to prevent and mitigate the pipeline failures. The list of main improvements needed as listed by his are; the need for a robust integrity management program, efficient ways to detect third party damage, and methodologies for better failure analysis of the oil and gas pipeline to understand the failure modes and mechanisms.

Salient Points: Robust Integrity Management Program needed**;** Third party damage is not detected**;** and Better Failure Analysis methodologies needed.

f) **Keynote Speaker: U.S. Department of Transportation (DOT), Pipeline and Hazardous Material Safety Division (PHMSA):** Robert Smith, Pipeline and Hazardous Material Safety Administration, Pipeline Safety, Raleigh, North Carolina.

Mr. Smith started his speech by covering the main aspects of the U.S. DOT, especially PHMSA research initiative. He stated that the pipelines are the safest mode of transportation and displayed statistics to support the claim. However, he stated that new models and tools are required to effectively asset manage the pipeline infrastructure systems. He concluded his speech by emphasizing the need for application of the intelligent sensor systems to detect the condition and the third party damages to the pipeline infrastructure systems.

Salient Points: Pipeline Safest Mode of transportation**;** New Models and Tools are required for performance prediction**;** Suite of intelligent and advanced sensors needed for pipelines**;** and Third party and excavation damage detection system needed.

g) **Keynote Speaker: U.S. Department of Energy (DOE) (Energy Pipeline Infrastructure System:** Christopher Freitas, Senior Program Manager for Natural Gas, U.S. DOE, Washington D.C.

Mr. Freitas discussed the role of U.S. DOE's on pipeline infrastructure research and development. He discussed DOE plans in short term (0-36 Months) and long term (3-5 years). He covered the research efforts by the U.S. DOE's Material Laboratory at Albany, NY. He concludes his speech by discussing some future research such as the liquid and gas quality sensors that can be embedded in the pipelines. He also discussed potential of future carbon pipeline infrastructure systems.

Salient Points: Short Term 0 – 36 and Medium Term 36 – 5 years Research and Development***;*** Future Pipeline - Carbon Pipeline***;*** Smart Pipeline should measure liquid and gas quality***;*** and Environmental Sensitivity and Security.

h) **Keynote Speaker: U.S. Congressional Research Service (CRS), Energy and Infrastructure Policy:** Paul Parfomak, Congressional Research Services, The Library of Congress, Washington D.C.

Dr. Parfomak discussed the social and environmental aspects of the pipeline infrastructure systems with the emphasis on the design, procurement, and construction process. He covered the Keystone Pipeline Extension (XL) project.

Why is Keystone XL such a big deal?

- Concurrent Events
  - Pipeline accidents
  - Mideast instability
- Developer Missteps
  - Assumed easy approval like prior pipelines
  - Local environmental concerns unexpected
- Political Environment
  - Global environment
  - Energy independence

He used Keystone Pipeline example to discuss the widening effect of environmental concerns and public influence on the pipeline projects. He concluded his speech on indicating emphasis on the importance of environmental impact analysis, public education, and outreach for success of the energy pipeline infrastructure projects.
Salient Points: Environmental Impact Analysis; Widening Environmental Scope**;** and Public education and outreach is critical.
i) **Keynote Speaker: U.S. Department of Homeland Security (DHS), Infrastructure Threat and Risk Analysis:** Marilee Orr, Homeland Infrastructure Threat and Risk Analysis Center (HITRAC), Washington D.C.
Ms. Marilee Orr mentioned that HITRAC assesses critical infrastructure risk to: identify infrastructure critical to the national's public health, economy, and national security; develop and manage risk analysis methodology and applied research; and respond to crises and real-time incidents by providing timely and actionable analysis for decision-makers. Ms. Orr also covered the nationwide modeling and simulation efforts of the Sandia and Los Alamos Laboratories. These efforts include the risk analysis of critical pipeline infrastructure, consequence analysis, and crisis action support for energy and water pipeline infrastructure systems.
Salient Points: All hazard approach modeling**;** Risk analysis of critical infrastructure, consequence analysis, and crisis action; Consider System of Systems approach for modeling and simulation**;** and Steady State and disturbed systems analysis.

TECHNICAL PROGRAM SUMMARY

To accomplish the overall goals, the workshop was conducted in five break-out sessions focused on five themes, each with its own goals and objectives, as briefly described under the five themes Performance & Durability, Sensing & Diagnostic, Installation & Retrofitting, Infrastructure & Society, and Education.

A. Theme of Five Technical Thrust Sessions:

a) Performance and Durability
b) Sensing and Diagnostics
c) Installation and Retrofitting
d) Infrastructure and Society
e) Education, Outreach, Training, Diversity, and Innovation Ecosystem

B. Purpose of Five Technical Thrust Sessions:

a) Update on the latest pipeline research and developments
b) Identify and document key research and educational issues
c) Identify and document major research and educational gaps
d) Identify short-term (5years) and long-term (10 years) needs
e) Prioritize research, educational, and outreach agenda

Technical Thrust Discussion

a) **Performance and Durability Thrust:**
Leader: Preet Singh, Georgia Tech and Moderator: Richard Thomasson, Arcadis
The Key Technical Issues Discussed in this Thrust is as follows:

- Holistic Design of Material and Research with objectives in mind
- Combined effect of defects and Inspection of joints and welds
- Standard Data Structure and Accelerated Testing is required
- Design for 300 years and Looking beyond hoop stress

- Probabilistic Design and Complex loading environment
- Multi-dimensional design criteria and Coating of Materials
- As built – baseline condition assessment
- Lack of good fundamental model, Multi-physic simulation model

b) **Sensing and Diagnostic Thrust:**
Leader: Irving Oppenheim, CMU and Moderator: Sam Cancilla, Redzone Robotics
The Key Technical Issues Discussed in this Thrust is as follows:
- Guided Wave Technology (need more details and reliability)
- Robotic Systems with multi-sensors, I&I, and leak detection technology
- CCTV inspection – data interpretation and use of Laser Technology
- Standardized Data collection, storage, retrieval, analysis, and update
- False positive and negative – Test-bed for reliability
- Fiber Optic Sensing and Signal processing algorithms
- Safety Culture – avoid liability and loss of revenue
- Operation System Interface – human factor

c) **Installation and Retrofitting Thrust:**
Leader: Erez Allouche, TTC and Moderator: Grant Whittle, Reline America
The Key Technical Issues Discussed in this Thrust is as follows:
- 3-D Mapping (more details and innovation needed)
- Third part damage is big issue for DOT – VDOT pilot project
- Novel Utility Location Technologies need (Multi-sensor approach)
- In-situ manufacturing (QA/QC need to be considered)
- New Material and Backfill (cross-cutting projects)
- Energy efficiency in installation and retrofitting
- Constructability Issues need to be considered

d) **Infrastructure and Society Thrust:**
Leader: Anne Khademian, Virginia Tech and Moderator: Walter Graf, WERF
The Key Technical Issues Discussed in this Thrust is as follows:
- Data for different framework – Asset Management
- Infrastructure is not sustainable, if it is not resilient
- Declining investment – it is true for all infrastructure
- Case studies should be both positive and negative
- Bond Rating and International Investors need to be considered
- Economic life and Physical Life (they are not same)
- Need to have an emphasis on consumers as part of the society component.

e) Education, Outreach, Training, and Innovation Ecosystem Thrust:
Leader: Robert McKim, Louisiana Tech and Moderator: Dennis Grove, VT
The Key Technical Issues Discussed in this Thrust is as follows:
- Community outreach and Changing workforce need to be addressed
- Web-based Database, Websites, and Social Media for outreach
- Modules for Manufacturing, Inspection, Construction, etc.
- Graduate Certificate program, and Public Engagement Program
- Engage Professional Societies, and Train new generation of engineers
- Vocational School and Industry Internship Program
- Service Learning & American Association of Community College

DISCUSSION SESSION RESULTS AND RESEARCH NEEDS OVERVIEW

Breakout Session 1: Performance and Durability – Discussion Summary

The goal of this thrust is to improve pipeline systems for safety, reliability, and cost-effective performance through improved understanding of failure mechanisms and performance criteria. This thrust will also develop novel materials and design concepts for both new and retrofitted pipelines with high performance, high reliability, and low life-cycle cost. Key barriers that will be addressed in this thrust are the present lack of understanding of the coupled effect and different failure modes and mechanisms for various classes of materials used in the pipeline systems.

Breakout Session 2: Sensing and Diagnostic – Discussion Summary

The goal of this thrust is to develop new sensing technologies, data collection and analysis algorithms, and wireless and energy-harvesting technologies capable of real-time, robust nondestructive evaluation (NDE) of pipeline infrastructure networks to improve their safety, security, and reliability, thus reducing catastrophic consequences of pipeline failures. This thrust addresses several barriers: (a) robustness (avoidance of false positives or false negatives) of sensing and diagnostics, (b) embedded and autonomous sensing in extreme conditions, (c) wireless power and communication methods at scale, and (d) life-cycle cost-effectiveness.

Breakout Session 3: Installation and Retrofitting – Discussion Summary

The goal of this thrust is to collaborate with all research thrusts and develop next-generation trenchless technologies, 4-D integrated real-time mapping systems for locating, inspecting, installing, and retrofitting pipeline systems, with minimal excavation and disturbance to existing infrastructure and the environment. This thrust will address key barriers in: (a) length, diameter, and depth of pipeline installed and retrofitted with minimum surface impacts; and (b) system of systems approach.

Breakout Session 4: Infrastructure and Society – Discussion Summary

The goal of this thrust is to research the interaction of economic, policy, environmental, social, governance, and private sector with the novel technologies and approaches developed by the Performance and Durability, Sensing and Diagnostic, and Installation and Retrofitting thrusts. SPINE Infrastructure and Society thrust addresses the creation of metrics to assess pipeline systems, recognizing that sustainability and resiliency are multidimensional and involve trade-offs among economic, policy, public health, environmental, legal, security, and social criteria. The research addresses several fundamental barriers: (a) the need to develop relevant theory to guide the integration of infrastructure and social priorities; (b) acquisition of data to create sustainability and resiliency criteria; (c) complex socio-economic, policy, and governance issues related to the development, implementation, and use of novel technologies, techniques, and approaches; (d) communication and engagement with stakeholders on the multidimensional effects of pipeline infrastructure components and system designs; and (e) the complexity of the pipe being modeled.

Breakout Session 5: Education, Outreach, and Training – Discussion Summary

SPINE's interdisciplinary educational and innovation program will be distinguished by its breadth and accessibility, a dynamic systems approach that is hands-on in its foundation, and the integration of social factors and public engagement. The educational programs will be designed with the objective of reaching students of all educational levels and professionals.

WORKSHOP SUMMARY

The Workshop was held to sharpen the SPINE vision and mission, both of which are focused on transforming the energy and water pipeline industry to make it sustainable and resilient. The participants represented a diverse cross section of researchers from academia, industry, and federal institutions. They identified resilient and sustainable pipeline infrastructure systems as the overarching goal.

The key topics discussed for energy and water pipeline infrastructure system research, education and outreach are:

- Pipeline workforce development and new generation of engineers
- Development of simulation models to predict pipeline performance
- Key barriers: governance, resource and financial scarcity, culture shift
- Need to recognize that there are technology risk and regulatory risk
- Stakeholders should be society, industry, financial, investors, and academics
- Regulations are different water and energy pipeline based on market structure
- Pipeline research and education barriers need to be aligned with projects
- Clearly define a process to reach out and engage pipeline stakeholders
- Center should provide opportunity and research can demonstrate benefits
- Management process is important – more tools in toolbox for decision-support
- Develop business model and financing - other than governmental support

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Developing Design Standards for a New Multi-Agency Regional Water Supply System

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Abstract

In a cooperative agreement, the Tualatin Valley Water District, its partners, and the cities of Hillsboro, Beaverton, Tigard, and Tualatin, Oregon have begun development of a new water system to provide greater redundancy and resiliency in a seismically active region. The new water supply as part of the Willamette Water Supply Program (WWSP), will take water from the Willamette River, and convey it to the terminal reservoir approximately 26 miles away. To move the water from the source to the connection points, nearly 35 miles of transmission main, ranging in size from 72 to 48 inches in diameter, must be constructed. Due to the size of the project, a set of common standards was developed; this will assure consistency between projects designed and constructed by different professionals, resulting in improved efficiency and easier maintenance and operations practices. Consensus was achieved by assembling drafts of design guidance sections, details, and specifications, and by hosting a series of workshops with technical and operations staff. This paper outlines the challenges faced and presents innovative techniques used to help ensure a long-term and reliable water supply to customers in the Pacific Northwest.

Introduction

The Willamette Supply system must be resilient and provide water to its customers when they need it, even after a major disaster such as a Cascadia Subduction Zone event. Yet, the benefits of each investment in reliability must also be balanced against the cost.

Design guidelines have been developed as a tool for the WWSP project to provide the standards that govern the design of a new transmission system.

The guidelines are divided into three sections:

- Design Guidelines;
- Standard Details and;
- Standard Specifications.

The WWSP goal is to provide a long-term, reliable, and resilient water supply to the region; the WWSP has prepared the *WWSP Design Guidelines, Standard Details, and Standard Specifications* documents to achieve its goals. These guidelines have been developed to provide a framework for the design of the many complex projects that will be part of the WWSP.

What is the purpose of guidelines?

The purpose of the guidelines is to provide uniformity in design and method for products made by future design consultants. Without guidelines, consultants for different pipeline construction design packages would use dissimilar construction materials, types of equipment, and individual design concepts. With the many types of materials and equipment that are the framework for projects of this size and duration, complications could arise associated with timely and predictable repairs after a seismic event, the storage of spare parts required for maintenance and repairs, and startup and operator training. Without design guidelines, owner review of construction packages becomes more difficult and inconsistent. The ultimate beneficiaries of the guidelines are the people and companies involved in design and construction, and the communities the pipeline will serve, as illustrated in Figure 1.

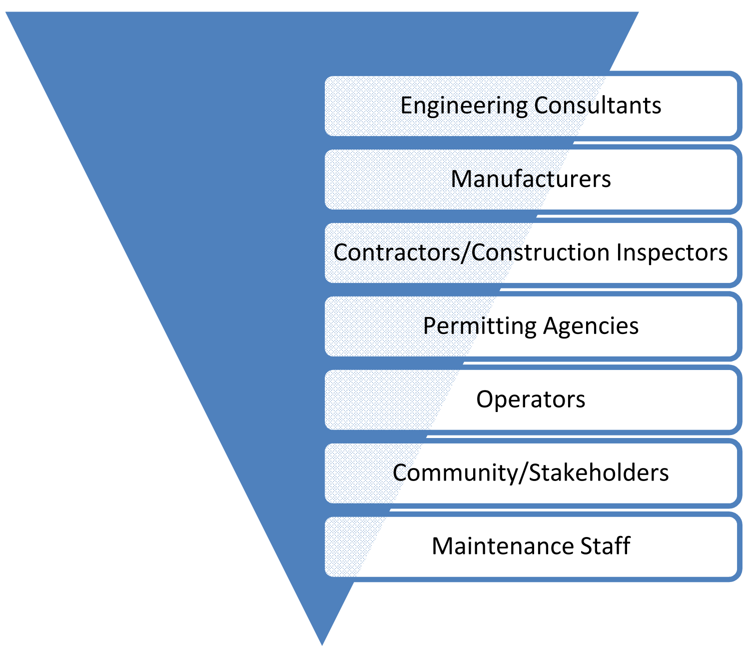

Figure 1. Guideline Beneficiaries

The purpose of the guidelines is not to repress creativity, inventiveness, or design innovations, but instead to assure design uniformity for standard conditions. Design Consultants will use these guidelines to design projects with approved concepts as part of the WWSP. Consultants are then responsible for each project's design efforts and legal obligations.

Guideline Development

The WWSP is a new organization made up of two different agencies, TVWD and the City of Hillsboro, each of which individually owns and operates water transmission lines, but neither has established large diameter (transmission line) design guidelines, details, or specifications prior to this design guideline effort. The Partners recognized the need to have guidelines due to the extended length of the project and multiple design consultants who would be contributing to the transmission pipeline design.

Developing design guidelines is not as simple as drafting details and writing specifications. Developing good design guidelines depends on a thorough understanding of the specific program and project needs, local conditions and level of service goals.

There were two components that drove the development of the WWSP guidelines in a positive and collaborative direction.

First, the new guidelines were developed from the "ground up", and based on the *AWWA Manual of Water Supply Practices* (M11), tailored specifically for the new program. The concept behind starting from the "ground up" is to meet one of the program's main objectives, to create a seismically resilient system. In order to do that, the Partners accepted a base design using AWWA M11 and built upon this basis by asking how the seismic resiliency of the pipeline system could be increased cost-effectively for the program. Components from pipe joints to air valve vaults were included in the guidelines as part of a seismic resilient system, not just individual appurtenances.

Second, the partners wanted a collaborative approach to guideline decision making so a series of workshops were conducted, with the focus both technical and collaborative. The goal of the workshops was to reach a stakeholder consensus on individual design and system elements that would be part of the guidelines.

Workshops

Given the nature of the project and the multiple agencies involved, it was necessary to obtain consensus of the partners regarding the design standards, details, and specifications. This was done through a series of eight technical workshops, illustrated in Figure 2 and Figure 3, each featuring discussion of best value design principles that would provide a cost effective, secure, dependable, resilient, and operable water supply system. Keeping these principles in mind, workshop attendees engaged in a series of small group discussions to identify, deliberate, and reach consensus on a basis of design for each design component.

The workshops were intentionally divided up so as to not overwhelm a single workshop with the introduction of the most technically difficult design components. For example, combination air valve assemblies were not introduced in the same workshop as mainline valves. Each of these components required lengthy technical discussion and having them separated helped with reducing meeting fatigue.

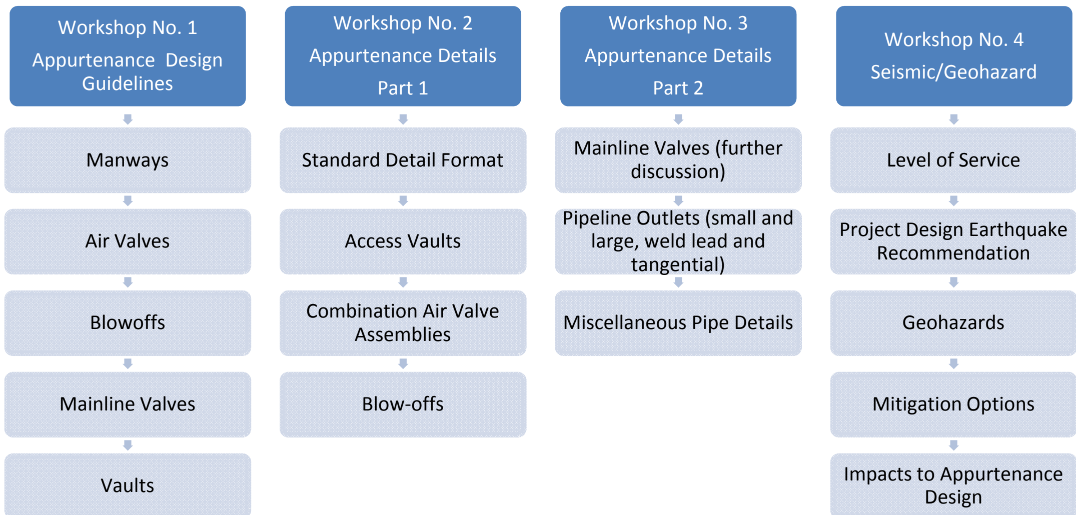

Figure 2. Design Workshops Part 1

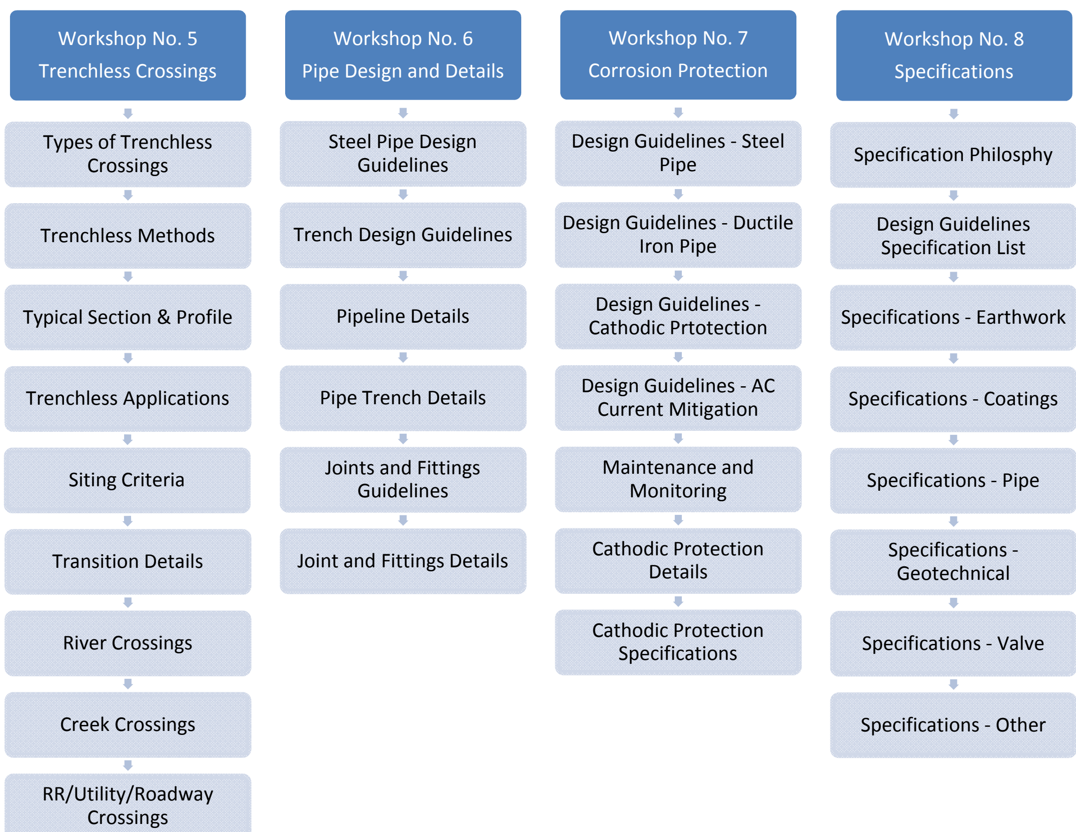

Figure 3. Design Workshops Part 2

Before the first workshops were conducted, the Partners' technical and operations staff provided input during prior project predesign tasks, which included technical memorandums on seismic and geotechnical design, about what was desired in certain aspects of the design. For example, was there a preference to have concrete floors in air valve vaults, or did they prefer air valve vaults over the top of pipe, or away from the pipe on the side of the road? With this input, a clearer and more established starting point was developed when a design component was presented at the workshops. The closer a design component, such as a vault, was to what the Partners' technical and operations staff envisioned, the sooner a productive conversation could be had, leading to resolution and adoption of the design component by the owner and staff. An example "roadmap" used in the pipeline appurtenance detail workshops is illustrated in Figure 4.

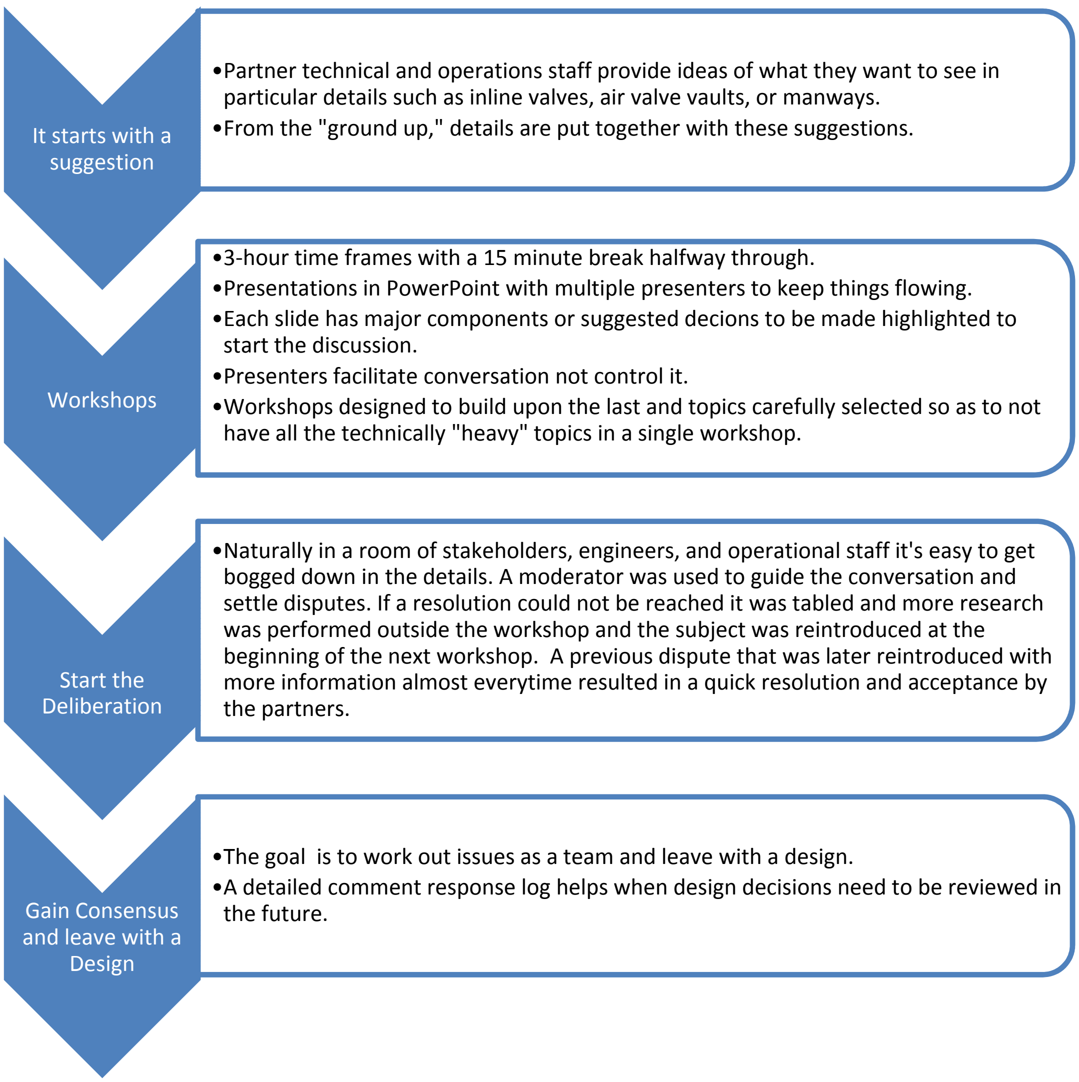

Figure 4. Detail Design Workshops Roadmap

Details

The following is two examples of the innovative ideas that came out of the workshops.

Combination Air Valve Vault Detail

The combination air valve vault is the largest pipeline appurtenance on the project and required the most discussion. Several workshops were required to determine its final form illustrated in Figure 5.

- The air valve is located on top of the manway. This provides a means to vent air that gets trapped in the manway which helps prevent corrosion and allows an unobstructed space, adjacent to the valve in the vault, to provide ease of access and maintain the valve.
- The air valve exhaust piping is connected to the air vent piping, leaving the vault with a flexible unrestrained coupling. This allows the piping to detach in a seismic event without applying large forces to the air valve. The air valve is hard-piped to the air vent above ground to prevent cross contamination with high ground water inside the vault.
- Although not shown in this sectional view, the vault has two 30-inch manways for surface access, with one centered over the air valve for easy maintenance access.
- The vault is not attached to the pipeline, which allows the vault to move independently from the pipeline during a seismic event. In addition, there is not a concrete floor; allowing surface and groundwater to move freely in and out of the vault.

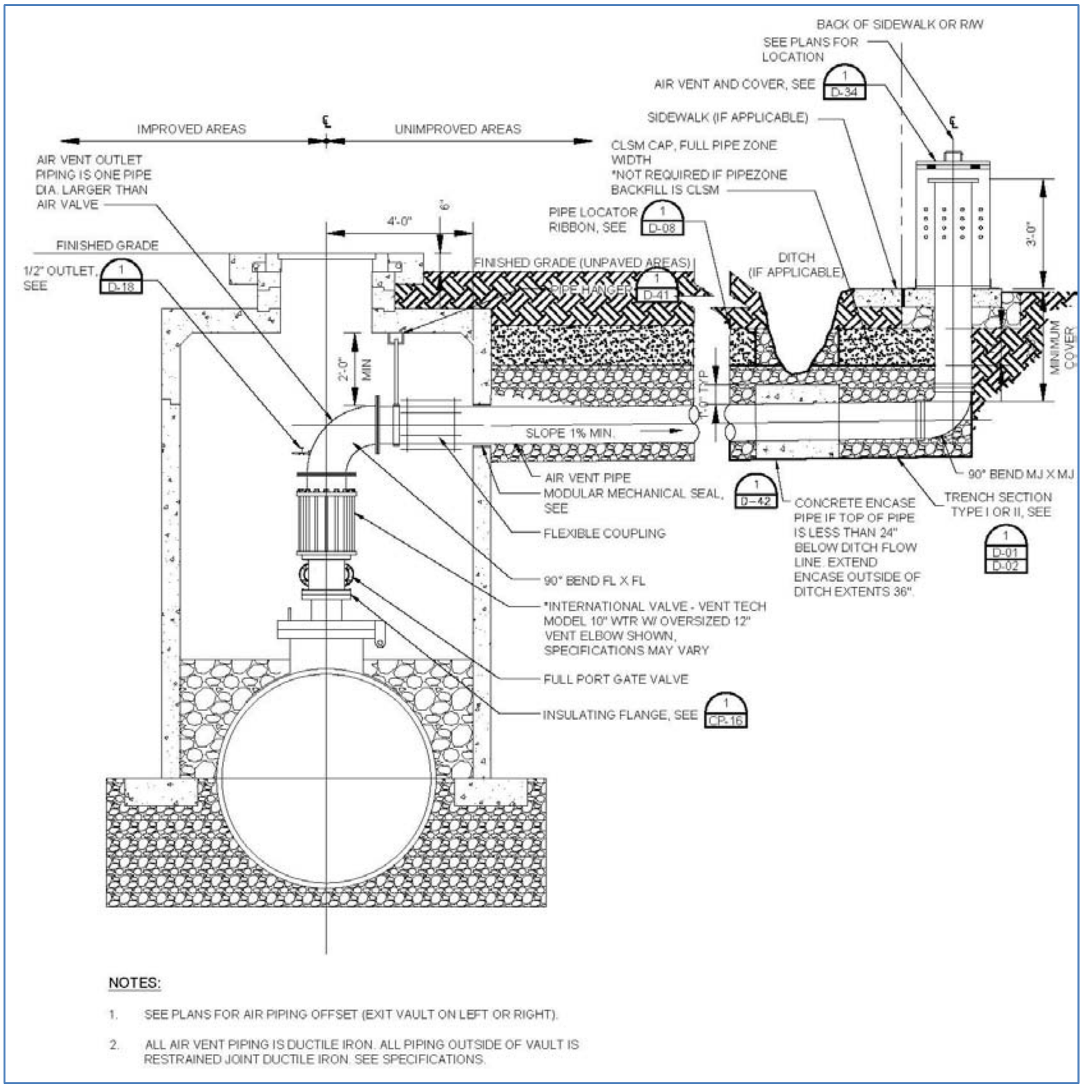

Figure 5. Combination Air Valve Vault

Blow-off

The blow-off is used to drain the pipeline during maintenance or emergencies. It took several workshops to develop it into its final form illustrated in Figure **6**.

- The only significant, resilient design element of the blow-off is the use of a force-balanced Flex-Tend between the isolation gate valve on the tangential outlet and the throttling butterfly valve. The force-balanced Flex-Tend allows 16.5 inches of offset, either side to side or up and down, which allows the large transmission pipeline to move independently of the adjacent blow-off piping and riser. The use of the balanced version of the force-balanced Flex-Tend mitigates thrust forces if a post-seismic event displaces the joint.
- In this blow-off assembly the owners wanted a design that has a controlled discharge where the rate of flow and direction could be controlled. The blow-off assembly has a gate valve connected to the transmission pipes tangential outlet for isolation and a butterfly fly valve for throttling flow. The isolation gate valve is for in case the butterfly valve seals are damaged during throttling. The blow-off discharge

outlet just below grade is a flanged connection where a flexible discharge hose will be connected and ran along the surface to a nearby ditch or storm water catch basin. The half-inch outlet on the blind flange is for connecting a pressure gauge in order to check for back pressure before beginning to unbolt the blind flange.

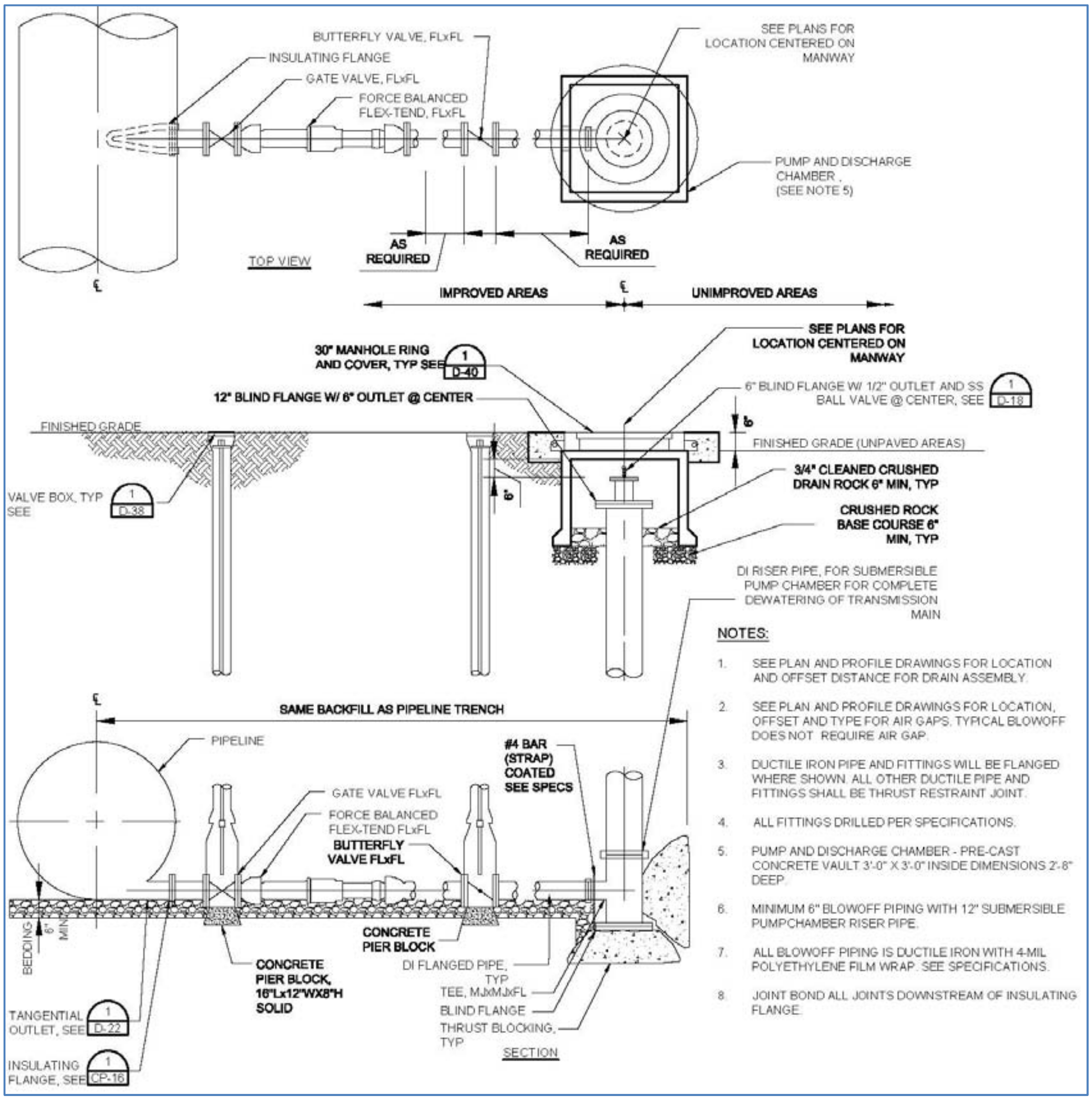

Figure 6. Blow-off

Current Status and Summary

The “ground up” approach used in developing the design guidelines for a new multi-agency regional water supply system was a success that all partners contributed to. The final design guideline documents have met the goal that everyone set out to achieve, of being the design “road map” for construction of a long-term reliable and resilient water supply system. A large part of that success is attributed to tailored workshops that proved to be engaging and an excellent forum for technical and operation staff to be heard, and for group consensus to be reached.

The first set of design guidelines are currently being used on the first 66-inch diameter transmission pipeline design package awarded to a consultant.

The guidelines are meant to be a living document. As such, consultants are encouraged to formally request deviations where physical constraints, hardships, or special conditions of the project prohibit meeting specific requirements of the design guidelines. Some of these deviations may result in permanent modifications to the guidelines. As feedback is received from innovative design consultants, the guidelines are sure to evolve through additional changes to meet project goals over the next decade of design and construction.

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Additional Information

Project website: http://www.ourreliablewater.org/

What Pipeline Management Can Do for You—A Review of the Costs and Benefits

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Abstract

Rather than implementing a traditional replacement program, several utilities are now opting to manage their pipeline assets using a more holistic approach with advanced engineering principles, condition assessment technologies, software, and multiple renewal strategies. Under this holistic approach, utilities are not only optimizing the life of the buried assets thereby reducing the capital costs of replacement, but also seeing other financial benefits. Many utilities under value their buried assets by simply stating the value of their systems based on typical straight-line depreciation. However, by implementing a more managed approach to their pipeline infrastructure utilities can increase their debt to equity ratio through recognizing the remaining life of their assets is greater than previously estimated in their existing depreciation model. This paper will present various analyses from holistic pipeline management programs providing several financial benefits over simple deferral of capital replacement. These analyses will focus on three main benefits: 1. The extension of the life of buried infrastructure; 2. prolong the costs of capital replacement and; 3. allow the utilities to determine a more accurate system value thereby increasing their financial standing.

DEFINITION/SUMMARY OF ASSET MANAGEMENT AS DEFINED BY THE INDUSTRY

Asset management is the systematic integration of advanced and sustainable management techniques throughout the organization. This is achieved by placing the primary focus of the organization on the long-term life cycle of the assets and their sustained performance, rather than on short-term, day-to-day aspects of the asset. Asset management insists on a culture of instituting industry best practices on every level of an organization's operation all while striving for continuous improvement. This definition, or some form of it, has been adopted by the US Environmental Protection Agency as well as many utilities throughout the world (Albee, 2009).

The primary goals of a utilities asset management program are:

- Maintain expected level of service for customers;
- Optimize and deploy human resources;
- Optimize life and value of physical resources;
- Minimize financial burden to customer.

To achieve these goals, a utility should attempt to answer the questions demonstrated in Figure 1.

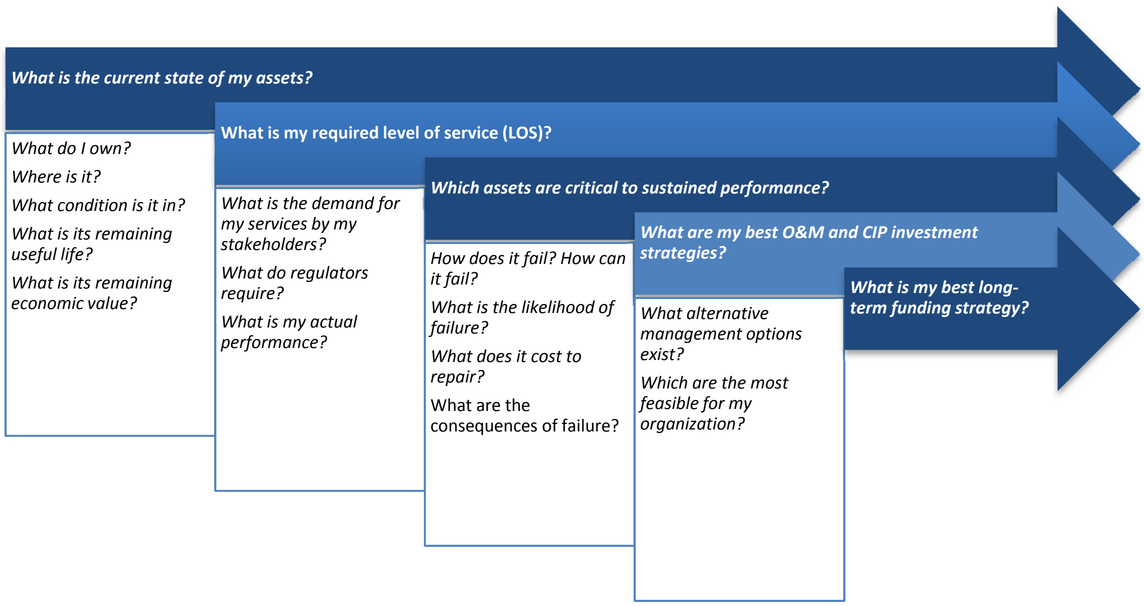

Figure 1. Fundamental questions for asset management programs

To answer these 5 questions, it is proposed a utility take a stepwise approach to building an asset management program with the desired outcome of implementation of the program with a focus on continuous improvement for each component. Figure 2 provides an overview of the stepwise approach to implementation of an asset management program also incorporating the 5 questions listed above.

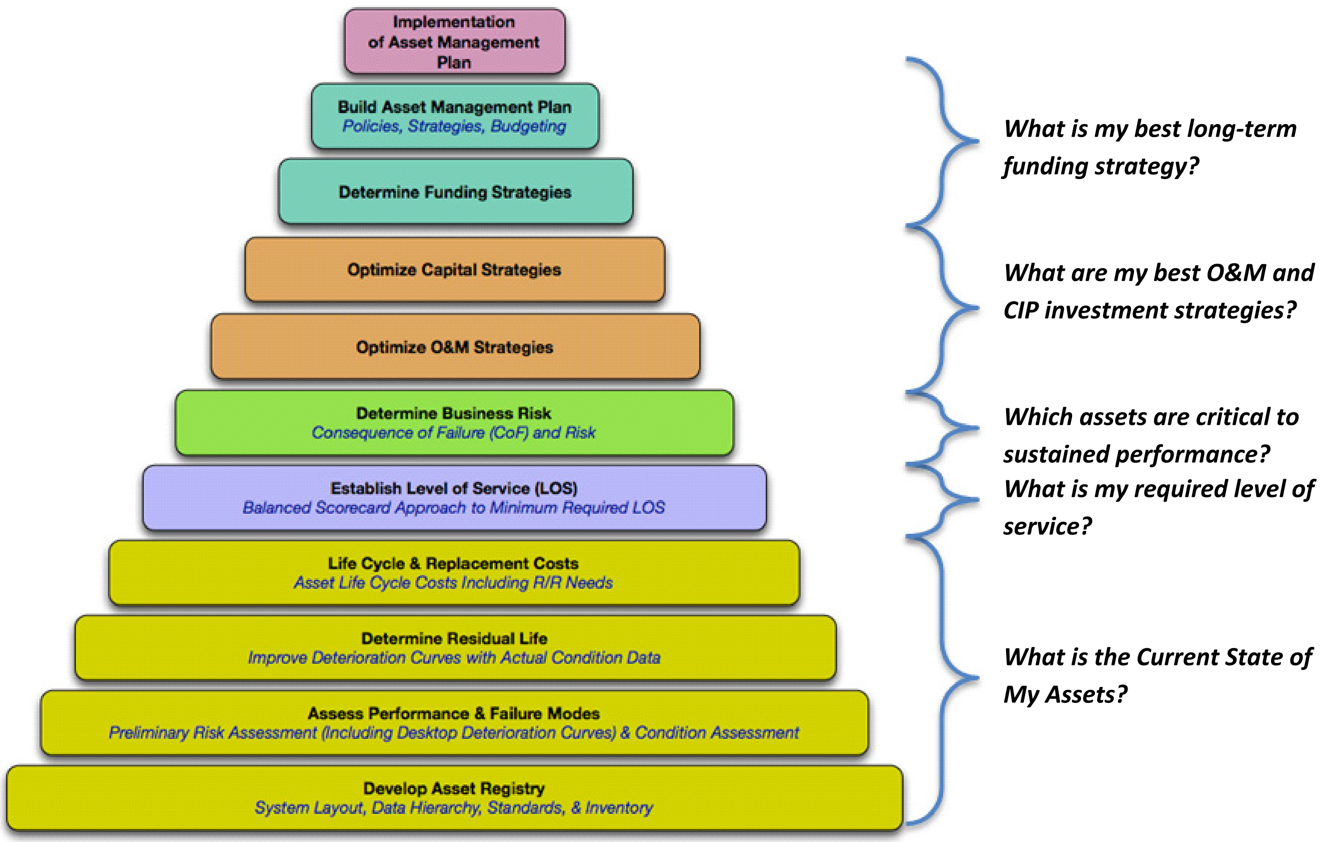

Figure 2. Stepwise approach to building an asset management program

CHALLENGES WITH APPLYING TRADITIONAL ASSET MANAGEMENT PRINCIPLES TO BURIED INFRASTRUCTURE

Although applying the approach outlined in Figures 1 and 2 to a utility's buried infrastructure has been performed for over a decade, it has been largely unsuccessful in meeting the desired level of service for a utility. Since gravity sewer management techniques have succeeded in data collection, evaluation, and renewal strategies; most utilities' focus has been largely weighted towards the non-pressurized pipe inventory (gravity sewer) due to relative ease in managing these pipes as well as regulatory pressure (Clean Water Act) compared to pressurized pipelines. Manageability of assets and regulatory drivers are certainly important elements in the decision making process for a utility, a more balanced approach is needed to properly manage the full buried infrastructure inventory (water, sewer, and storm water). This balanced approach has recently been recognized by the City of Baltimore, Maryland and approved by the US Department of Justice in order to allow the City to effectively target its entire infrastructure, not just sewers.

While as previously noted, gravity sewers and to some extent, storm sewers have benefited from accessibility and applicability of visual techniques for inspection, pressurized pipeline management has been significantly more difficult. Until recently, basing pipeline renewal on desktop risk prioritizations has been the only available approach to utilities for planning and managing pipeline replacement. These risk assessments typically use age and the expected life (book value) of an asset to determine the remaining service life of pipelines, which is then a significant factor in the replacement strategy. However, age has been shown to be one of the least reliable predictive factors in pipe failure based on findings by the US EPA, Water Research Foundation, and multiple utilities where data indicates that 70% to 90% of the replaced pipe has remaining life (Albee, 2009). This realization is causing a shift in industry attitudes away from this *Traditional Approach*.

Using asset risk to guide the management strategies, owners can ensure they are implementing the right approach, at the right time, with the lowest financial impact. While recent advances in pressure pipe inspection technologies, assessment techniques, and repair/rehabilitation methods now allow for substantial extension of existing asset service life, a risk based approach to their implementation will ensure resources are focused on the correct pipelines. The goal should always be to focus the proper resources in managing the asset while safely getting the most service life out of the pipeline.

While information presented in this paper will deliver general asset management principles, it should be noted that the primary focus is on the management of pipeline infrastructure.

DEVELOPING THE ASSET REGISTER

Many publications have focused on techniques and strategies for establishing the asset register for a utility therefore the authors will not focus on these topics in great detail. However the following should be noted and/or considered:

- An asset register should facilitate the systematic recording of all assets a utility is responsible for;
- A unique identifier should be established (numerical ID) for each asset where any attribute information can be linked;
- An owner/manager of the asset should carefully consider to what level an asset will be managed prior to finalizing a hierarchy;
- Assets of the same component type (e.g. pumps, valves, pipes) may have differing useful lives that must be accommodated;
- Costs and maintenance activities must be apportioned to the most granular level of the hierarchy.

With respect to the third bullet above, definition of an asset is critical in establishing proper and realistic management strategies. This is especially true for pipelines. For example, the length of pipeline to be defined as an asset plays a critical role in asset management practices. This could differ in granularity based on the utilities approach. Pipelines may be managed from valve to valve, by contract (contract number of installation), or even pipe stick to pipe stick depending on the data resolution and quality available. As the asset hierarchy is developed, these items must be considered. A utility should not feel constrained in selection of the asset definition however as even if a pipeline is managed on a contract level (up to miles in length) but data collected on a more granular level. Pipeline data can easily be aggregated to the desired management length in order to provide a robust risk value.

USE RISK TO DRIVE DATA COLLECTION

As pipe diameters decrease, the applicability of inspection technologies and assessment techniques become more limited from a practicality and cost benefit perspective. Therefore, efforts should focus on collecting valuable information in the most efficient and cost effective manner possible. Prior to the selection of inspection and assessment techniques for pipeline infrastructure, an understanding of the failure modes for each pipe material of interest must be developed. The management approach should take into account the most common failure modes for all pipeline materials. Corrosion, physical stresses (i.e. overloading, seismic, pressure fluctuations, thermal stresses, etc.), operations, maintenance, and inherent material or installation flaws can work independently or in conjunction and lead to pipeline failure. These deterioration methods often progress over years beginning with pipe wall loss and leaks in relatively small locations, potentially expanding to widespread corrosion, cracking, and/or failures.

Data collection and root cause analysis has been collected for a significant number of small to large diameter water main failures over the years. An example of a multi-year effort for a large utility consisting of onsite documentation of all water main breaks 6-inches in diameter and larger is provided below as Figure 3. The figure provides a snapshot of the data collected and subsequent root cause analysis. Based on these findings, the utility has decided to use traditional correlator leak detection technologies as a management approach (performed by in house staff) for its small diameter water mains over other, more costly tools.

Data Collected

- Over 80 Leaks & Breaks
- Materials - Cast Iron, Ductile Irons, PVC, & Steel
- 6-inches to 24-inches in Diameter
- Installation Years Between 1949 and 2007

Findings

- 86% of Ferrous Pipe Exhibited Graphitic Corrosion
- 55% of Failures Preceded by Long-Term Leakage
- 21% of Failures at or Adjacent to Copper Service Taps
- 3% of Failures Adacent to Previous Repairs (Repair Clamps)
- None of the Failures Showed Significant Internal Corrosion

Figure 3. Data collection and root causes analysis results for a large utility

As noted by the results presented above as well as information collected from utilities across North America, failures are often preceded by leaks. This information is important when determining the most suitable and cost effective inspection strategies for utilities. Another take away from these forensic studies is the inability to extract true root cause of failure data from the break records. Many utilities do not train or equip field crews to collect minimum data required to define the cause of a failure and to determine what conditions led to the break or leak. This may lead to several repairs being conducted on the same water main within a few feet of each other thereby skewing the risk model based on only a small area of the pipeline. Utilities can remedy this problem by adequate training of field staff to collect baseline information for each repair using a standard hard copy or electronic templates.

Taking pipe material specific failure modes into account, a risk based assessment strategy, similar to the one shown in the Figure 4 below, can then be performed. It should be noted, that while this diagram can provide general guidance for a utility's buried infrastructure management program, more information will be required to finalize the risk based condition assessment approach including incorporation of a risk ranking system. For the purposes of this proposal, the authors have broken the various risk categories into three risk categories – low, medium, and high. This may be revised based on the final risk ranking system developed. While the approach presented herein provides insight into possible assessment techniques for a water system, the goal is to develop a programmatic approach that is flexible enough to incorporate additional technologies, analyses, and data that may not be specifically proposed.

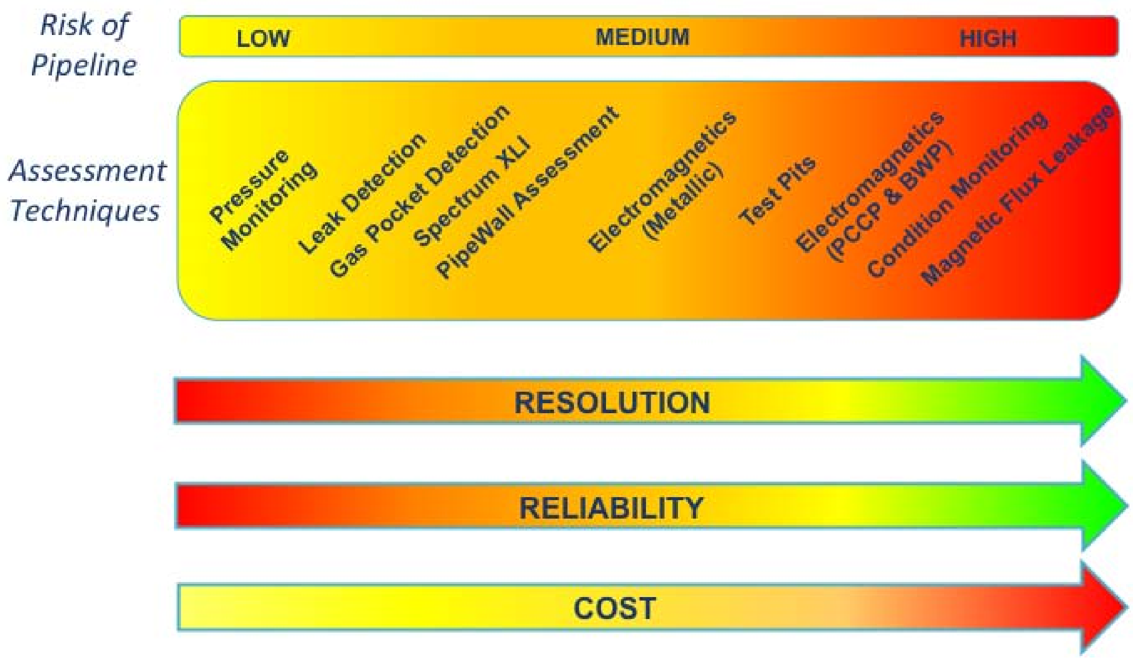

Figure 4. Risk based asset management strategy

Structural Analysis

After collection of the risk based condition data, structural models are important as they provide insight as to the true condition of the pipe rather than just reported anomalies. **Deterioration does not necessarily indicate a pipe requires immediate or short-term renewal.** An example of a condition model is illustrated in the Figure 5 below. This curve represents the Failure Envelope for an 8-inch cast iron pipe (CIP) (structural models for asbestos cement pipe (ACP) are nearly identical with the exception of pipe attribute inputs). The failure envelope is calculated by combining the external loading (y-axis) with internal loading or pressure (x-axis). Should the stresses exceed the failure envelope, failure of the pipe will likely occur. Recently developed structural models have taken this model a step further than simply providing the design analysis by allowing for the inclusion of corrosion (pipe wall loss) data. Specifically, the figure uses the following assumptions:

- 8-inch spun cast iron pipe designed under the AWWA C108-62 standard (Class 22 with a nominal wall thickness of 0.41 inches);
- Minimum of 5 feet of cover with a maximum of 8 feet of cover;
- Operating pressure of 75 PSI with a surge allowance of 120 PSI;
- Standard traffic loading;
- Localized corrosion area with a 50% loss in thickness over an area 6-inches in width by 12-inches in length.

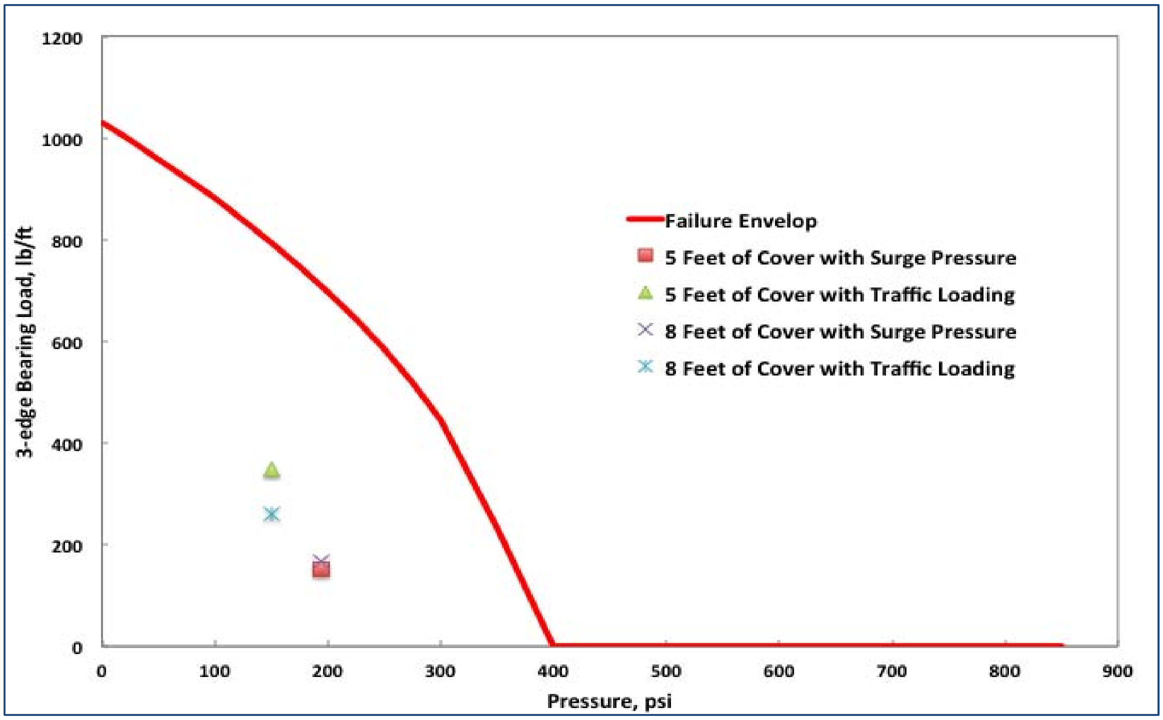

Figure 5 – Example Structural Model for Cast Iron Pipe

As can be seen in the Figure 5, despite having 50% wall loss over such a large area, the pipe still does not exceed the failure envelope at the given parameters. In fact, the pipe does not reach the failure envelope until almost two-thirds (65%) of the pipe wall has been lost. Combining structural analysis with the inspection data collected as part of a comprehensive assessment approach can be a powerful tool in extending the life of the existing pipelines, even if deterioration has taken place.

Life-Cycle Analysis

By combining the structural analysis with condition data, estimates of when the pipeline should next be inspected along with a remaining service life of the asset can be completed. To do this, statistical simulations may be used that incorporate failure history, inspection data, and structural analysis. An example of the output of this model is shown in Figure 6 below. The number of failures predicted (y-axis) by year into the future (x-axis) is shown. Note that failures predicted in this case are not ruptures, but occurrences of pipe wall deterioration where the risk at the current operational level is unacceptable based on the structural analysis. The simulation data indicates that based on the data collected, no failures are expected to occur prior to 2035, which is the assumed end of life for the asset and where the utility had planned for replacement. However, it can be seen that estimated failures are not expected until 2055, 20 years past the assumed end of life and an 85% survivability is expected through nearly 2090. Based on these results, the pipeline may remain in service for a substantially longer period, therefore have a higher asset value, and defer capital expenditures.

It should be noted however that remaining useful life estimates should be used as guidance for re-inspection interval planning as collection of subsequent condition data can be used to better refine the asset life estimates. Once another inspection is completed, the data collected in that inspection should be analyzed in conjunction with the data presented in this report to provide a more accurate and robust remaining useful life evaluation. In the example below, re-inspection was recommended within 10 years from the original data collection (based on condition, failure history, and budget).

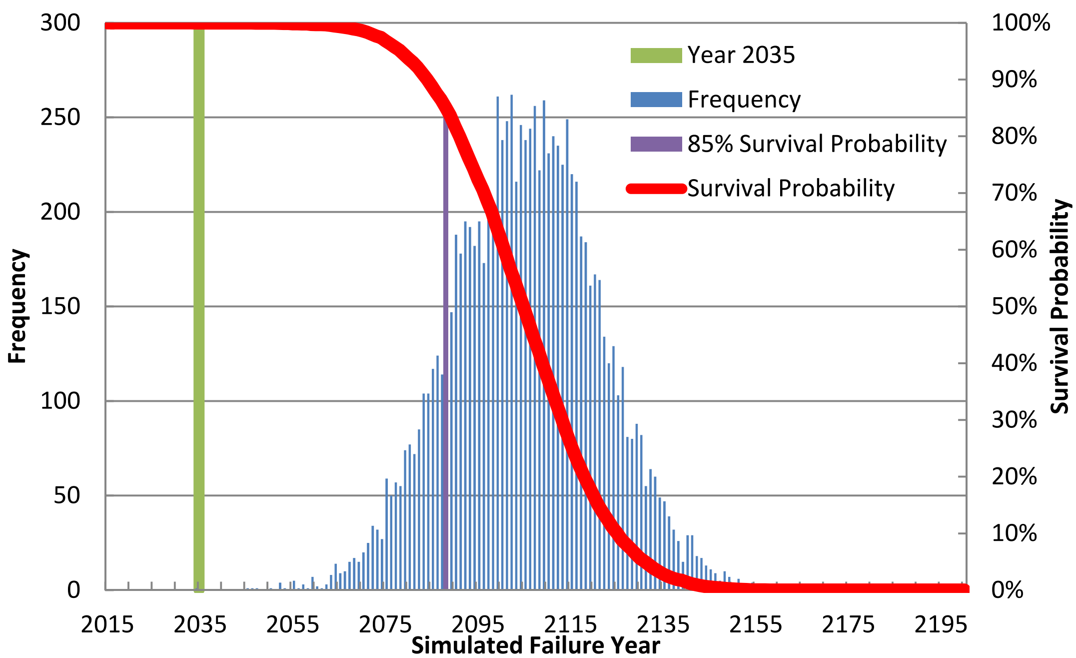

Figure 6. Example output of statistical simulation of failures

Collecting the Right Data

Selecting the correct assessment and analytical methods are crucial in a long-term water system management strategy. Selecting too conservative of an approach regarding assessment techniques may cause overspending on both the assessment and subsequent rehabilitation and repair. Alternatively, using too high of a resolution of assessment technology on low risk pipes may lead to overspending on the inspections with diminishing returns with respect to rehabilitation or replacement savings. This can be seen in comparing a soil screening technique (non-invasive) versus direct assessment (invasive) through test pits. Corrosion rates for DIP based on the screening technique versus the pipe wall thickness measurement results (test pitting) are shown in Figure 7. Comparing the two data sets below, the normal probability density functions for each data set indicate little correlation. The pipe wall measurement data set (invasive test pitting) produces a density function with relatively small variance while the soil testing data set (non-invasive screening) has a large variance. The means of each data set also have significant separation. The large variance in the soil testing data is likely due to a large variation in soil corrosivity along the pipe alignment while the small variance in the pipe wall measurement data indicates that the level of external corrosion at the sample locations is relatively consistent. It should be noted that the test pits aligned with areas where high corrosion rates were predicted by the corrosion analysis. These results highlight the significance of collecting higher resolution data for a high risk asset as the lower resolution data not only predicts failures earlier than the direct measurements but it also has a lower confidence level as it relates to the reliability of the information. Specifically, the corrosion rates based on the soil data indicates the time to failure can range from anywhere between 6 years through 42 years based on the variance in the normal distribution of the data. Time to failure rates based on the invasive, test pitting method began at 25 years.

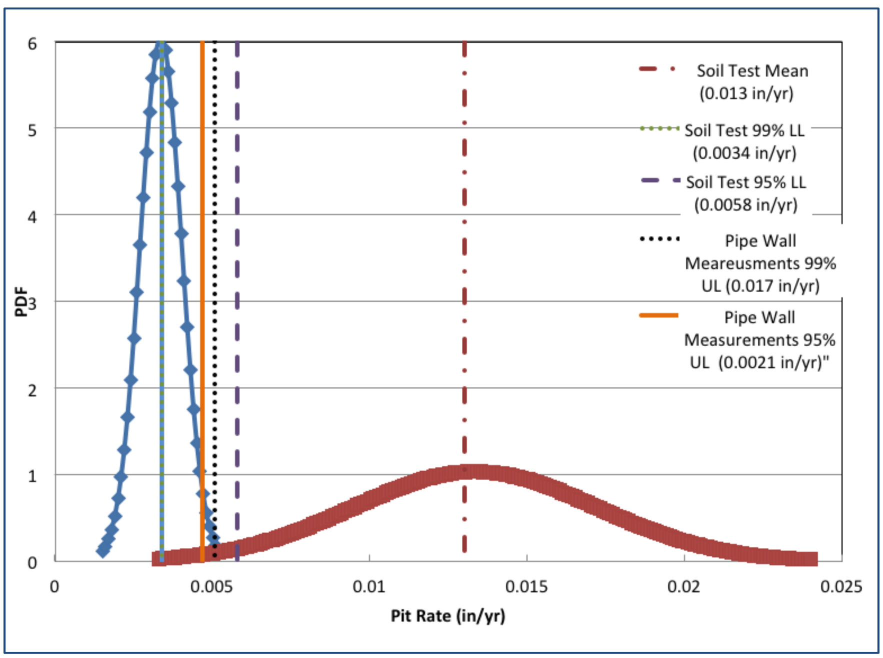

Figure 7. Corrosion rates for DIP based on the screening technique versus the pipe wall thickness measurement results (test pitting)

VALUE OF COLLECTING CONDITION DATA FROM RISK RATHER THAN REPLACEMENT

In order to establish whether a risk based management approach rather than systematic replacement is performed, the long-term financial viability of each strategy should be evaluated. This evaluation may include several management approaches including a do nothing (run to failure), systematic replacement based on some form of desktop risk assessment, and implementation of a management program that identifies and addresses pipe sections in need of replacement/repair on an as needed basis.

The example management strategies described below were developed using a utility's historical pipeline information, data collected as part of inspections, estimated financial data for failure costs, and estimated replacement costs for the pipelines from a specific utility's inventory. All of these costs were evaluated for two previously noted management strategies (systematic replacement and a management program) for a 20-year future projection. By utilizing a 20-year evaluation period, standard assumptions can be made when comparing the financial impacts of the management strategies while accounting for an average pipeline life of 50 to 75 years. As the majority of the system installed between 1965 and 1995, several of the pipelines would be programmed for replacement based on their age and expected life.

The financial modeling for the systematic replacement approach started at the beginning of a replacement investment cycle (2015). The annual investment needs were developed by forecasting future replacement needs based on a desktop risk assessment, the cost for replacing the old pipe, and other associated engineering and construction costs. Replacement cost per foot of pipe was based on recent bid tabulation sheets provided by the utility. No maintenance costs were included in the replacement projection.

The modeling for the management program considered an input for annual inspection of the same length of pipe as proposed for replacement. Repair costs for the management program were assumed to be 12.5% of the systematic replacement costs on an annual basis. This value was derived from an estimate of several pipeline management programs across North America including both small and large diameter pipe. An additional 20% annual contingency was added to the management costs for conservancy.

It should be noted that it is important to evaluate the full risk of a pipeline using both the likelihood (condition) and consequence (criticality) of failure to arrive at the optimized management strategy. Because consequence of failure is utility dependent with a high degree of subjectivity, this analysis focused on the likelihood of failure. Monetization of the consequence of failure is currently under development for this utility and will be managing using a complete risk based approach.

Table 1 summarizes the 20-year results for the systematic replacement versus a pipeline management program. This analysis focused on minimizing the overall capital expenditures on either the systematic replacement or management program options while maintaining the same operational expenditures. These operational expenditures account for failure repairs only. While the operational expenditures for both options increase at the same rate, the pipeline management option provides the same operational budget results (and associated predicted pipe failure rate) as the system replacement of pipes. This provides the utility with a potentially significant savings over 20 years.

Table 1: 20-Year Cumulative Costs for Pipeline Management Approaches

Approach	20-Year Cumulative Cost (Millions of Dollars)
Systematic Replacement	$122.4
Management Program	$28.8
Operational Costs of Failures	$52.8

CONCLUSIONS

Asset management programs for water and wastewater assets have been in development for over two decades across the globe. The concepts and implementation have proven successful for a utility's vertical assets (plants, pump stations, etc.) but have been challenging for the buried infrastructure, especially the pressurized pipelines (water mains and wastewater force mains). Varied levels of success have been achieved with the gravity sewer systems due in large part to asset accessibility (manholes) and ability to apply simple and repeatable assessment techniques (visual inspection). However, pressurized pipe systems have focused on using desktop assessments to prioritize replacement with little focus on collecting actual data on the assets. US EPA has estimated that 70% to 90% of the pipelines replaced have remaining life left in them due to this approach. Given the significant cost of replacing or rehabilitating pipelines, a more cost effective approach is needed.

By evaluating the overall risk of the pipeline assets when compared to the utility's asset inventory as a whole, a data collection (inspection) program can be established to determine not only the condition of each asset (vertical or horizontal) but allow the owner to more cost effectively maximize the useful life of the system component. The goal is to use risk (likelihood and consequence of failure) as a guide for not only what assets to inspect first, but select the appropriate assessment technique based on risk. The goal being to put the right amount of money towards the most appropriate asset at the right time. This provides a utility with an effective and defensible approach to managing their assets, it actually defers long-term funding needs by maximizing the life of an asset. This can be seen with an example provided for a utility serving a population of approximately 250,000. In this case, a risk-based management approach for performing condition assessment of buried infrastructure rather than simply replacing the assets based on a desktop risk model can save the utility over 75% of the replacement costs. These results are similar for other utilities across North America and the approach can be applied to all assets, not just the buried infrastructure.

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Developing a Pre-Certification Process for Using ISI Envision during the Planning Phase of a Pipeline Project

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Abstract

The Willamette Water Supply Program (WWSP) has teamed with the Institute for Sustainable Infrastructure (ISI) to develop a "pre-certification" process for the Envision™ Sustainable Infrastructure Rating System (Envision). The "pre-certification" process is being used during the planning phase of a major water supply program to inform evaluation of pipeline routes, selection of reservoir sites, and development of design guidelines to provide a sustainable water supply project that fits well in the community. This first-of-its-kind approach will use Envision during the planning and preliminary design phase in order to expand the focus of the engineering team to consider criteria beyond just engineering. The WWSP has incorporated the Envision rating criteria into the process for determining the pipeline route and reservoir site location and for developing design guidelines. Envision criteria are being used to minimize disruption to the surrounding communities and design a sustainable infrastructure project.

INTRODUCTION

The WWSP will provide an additional redundant, resilient, reliable water supply source for more than 400,000 people in the Portland, Oregon metropolitan region. The program, which is being jointly-developed by the City of Hillsboro, and the Tualatin Valley Water District (TVWD), will withdraw water from the Willamette River near Wilsonville, Oregon and deliver the water to two locations approximately twenty-four miles (38 km) north of the river.

Program components include improvements to the existing intake facility at the Willamette River Water Treatment Plant near Wilsonville, Oregon; approximately thirty-two miles (51 km) of large diameter transmission pipeline; an expanded water treatment plant near the existing Willamette River Water Treatment Plant; and a new thirty million gallon (113,000 m^3) enclosed terminal reservoir storage tank. The program will construct a new 72-inch (1.8 m) transmission pipeline from the treatment plant to the reservoir and pipelines that vary in size from 72-inch to 42-inch (183 cm to 105 cm) between the reservoir and existing distribution systems east and northwest of the reservoir site.

The first construction project in the program begins in 2016. The water supply system will be placed in service in 2026.

Figure 1. Willamette Water Supply vicinity map.

A Legacy of Sustainability and Conservation

The City of Hillsboro and the Tualatin Valley Water District have a long history of integrating sustainability into their programs to supply clean reliable water to their customers.

Both agencies are founding members of Partners for a Sustainable Washington County Community (PSWCC) and both agencies have been recognized locally and nationally for their conservation measures and other sustainability, leadership, and communications programs that lessen their impacts on the environment and the communities.

Collectively, they have won awards for exceptional management and leadership such as The Association of Metropolitan Water Agencies (AMWA) Platinum Award for Competitiveness Achievement and the Pacific Northwest Section - American Water Works Association (PNWS-AWWA) Outstanding Leadership and Support by an Organization Award. They have won sustainability and conservation awards such as American Public Works Association (APWA) Julian Prize for Sustainability. They have also won awards for engineering excellence from American Council of Engineering Companies (ACEC) and PNWS-AWWA. In addition, the TVWD facility earned a Silver Certification from the U.S. Green Building Council Leadership in Energy and Environmental Design (LEED).

TVWD staff helped review the early versions of Envision as it was developed by the American Society of Civil Engineers (ASCE), American Public Works Association (APWA), and American Council for Engineering Companies (ACEC).

When the two agencies teamed in late 2013 to develop the mid-Willamette River as a new source of drinking water, they identified the Envision™ Rating System (Envision) as a process that might help ensure a more sustainable project and that could help demonstrate to the public that the program is indeed considering how it impacts and benefits the surrounding communities.

The Envision™ Rating System

Envision is a project assessment and guidance tool, which was created to evaluate, grade, and give recognition to infrastructure projects that contribute to a more sustainable future. The stated purpose of Envision is to transform the way infrastructure is designed, constructed, operated, and maintained (Source: ISI 2012).

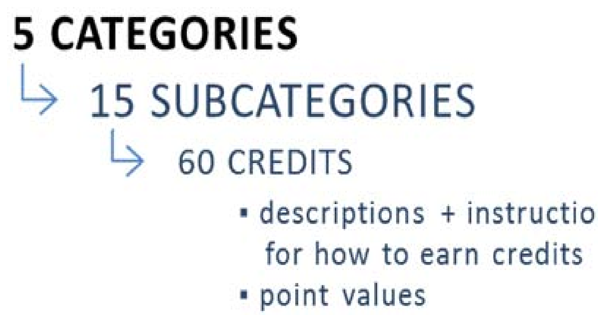

Figure 2. Envision credit rating framework (Source: ISI 2012)

Envision was developed collaboratively by the Zofnass Program for Sustainable Infrastructure at the Harvard University Graduate School of Design and the Institute for Sustainable Infrastructure with the goal of providing a comprehensive rating system that can be used to evaluate civil infrastructure projects and their effect on the environment, communities, economic growth, and public health.

Envision considers sixty criteria grouped into five major categories listed in Table 1 to address economic, environmental, and social implications of infrastructure projects on the surrounding community and environment. Envision is intended to address sustainability for roads, bridges, pipelines, railways, airports, dams, levees, landfills, water and wastewater treatment systems, and similar civil infrastructure projects. It is not intended for buildings or facilities that are covered by other sustainability rating systems such as LEED™ (Source: ISI 2012). Table 1 identifies the five Envision Categories and their corresponding subcategories.

Table 1. Envision Categories and subcategories (Source: ISI 2012)

Category	**Subcategories**
Quality of Life:	Purpose, Community, Wellbeing
Leadership:	Collaboration, Management, Planning
Resource Allocation	Materials, Energy, Water
Natural World	Siting, Land and Water, Biodiversity
Climate and Risk	Emissions, Resilience

The Envision credit rating system is used to evaluate projects by reviewing the project against each of the 60 individual credits. For each credit, the project reviewer verifies that their project meets the minimum standards required to obtain a particular Achievement Level. Achievement levels are the five point scale developed by ISI to measure how well a project fits each Envision credit.

Figure 3 illustrates the Achievement Levels and the relationship between levels for one of the twelve credits considered for the Quality of Life category. The top row in the table lists the available Achievement Levels – *Improved*, *Enhanced*, *Superior*, *Conserving*, or *Restorative*. The second row lists the points available if the project meets the minimum requirements for an Achievement Level. The WWSP program meets the requirements for *Conserving* and earns 20 points toward the overall Envision score.

Numeric scores are assigned for each Achievement Level. The scores for Achievement Levels vary among credits, but the order of applying Achievement Levels to each credit is consistent for all credits. Achievement Levels always build on each other. It is not possible to achieve a *Superior* level and obtain the accompanying credits if the project does not meet the requirements of Enhanced and Improved Achievement Levels.

In order to determine the highest possible Achievement Level, the project reviewer verifies that their project meets the criteria listed below each Achievement Level in Figure 3.

The WWSP project reached an Achievement Level of *Conserving* for this rating category. In order to achieve that rating the project also met the criteria listed under *Improved*, *Enhanced*, and *Superior* as denoted with a checked box in each Level.

In some cases, the requirement to meet the level of achievement would only apply to that level and would not necessarily apply to higher levels due to the way the requirement is worded. For example, in Figure 3, two of the four criteria for the *Improved* level would only apply if the *Improved* level was the highest Achievement Level possible for this project.

By considering sixty sustainability credits throughout the project development process (planning, design, construction, and operation), Envision provides a holistic view of how a project really fits into the surrounding environment and how the project impacts or benefits its neighbors and the community.

Maximize Envision Benefits

Envision views sustainability holistically. It incorporates environmental, social, economic impacts and benefits of a project to make sure the right project is being built the right way. It incorporates a self-verification process and a third-party review to verify that the project truly meets the high standards set by the Envision Sustainable Infrastructure Rating System.

QL 1.1 Improve Quality of Life

Intent: Improve the net quality of life of all communities affected by the project and mitigate negative impacts to communities.
Metric: Measures taken to assess community needs and improve quality of life while minimizing negative impacts.

LEVELS OF ACHIEVEMENT

IMPROVED	ENHANCED	SUPERIOR	CONSERVING	RESTORATIVE
(2) Internal focus. ☑ The project team has located and reviewed the most recent and relevant community planning information. ⊘ Some, but not systematic, outreach to stakeholders and decision makers has taken place. ⊘ Some relatively easy, but not particularly important or meaningful changes made to the project. ⊘ No significant adverse community effects are caused by the project. (A, B, C)	**(5) Community linkages.** ☑ More substantive efforts to locate, review, assess and incorporate the needs, goals and plans of the host community into the project. ☑ Most potential negative adverse impacts of the project on the host community are reduced or eliminated. ☑ Key stakeholders are involved in the project decision-making process. (A, B, C)	**(10) Broad community alignment.** ☑ All relevant community plans are reviewed and verified through stakeholder input. ☑ The project team works to achieve good project alignment with community plans, recognizing that the scope of the project is a limiting factor. ☑ Potential negative impacts on nearby affected communities are reduced or eliminated. (A, B, C)	**(20) Holistic assessment and collaboration.** ☑ The project makes a net positive contribution to the quality of life of the host and nearby affected communities. ☑ The project team makes a holistic assessment of community needs, goals and plans, incorporating meaningful stakeholder input. ☑ Project meets or exceeds important identified community needs and long-term requirements for sustainability. ☑ Remaining adverse impacts are minimal, mostly accepted as reasonable tradeoffs for benefits achieved. ☑ The project has broad community endorsement. (A, B, C)	**(25) Community renaissance.** ☐ Through rehabilitation of important community assets, upgraded and extended access, increased safety, improved environmental quality and additional infrastructure capacity, the project substantially reinvigorates the host and nearby communities. ☐ Working in genuine collaboration with stakeholders and community decision-makers, the project owner and the project team scope the project in a way that elevates community awareness and pride. ☐ Overall quality of life in these communities is markedly elevated. (A, B, C, D)

Figure 3. Envision credit rating Achievement Levels

One of the major benefits of using Envision to review and "rate" a project is the third-party verification by ISI, which can increase the credibility of the project with the public, elected officials, and other stakeholders. It is one thing to claim to have designed a sustainable, low-impact, beneficial project. It is much more powerful to have a third-party review the project and verify that you did the right project the right way based on a thorough review of the project against established criteria.

On WWSP, Envision was incorporated into the Program Values that were developed by the Public Involvement Team. The values were used to set baseline expectations when communicating with the communities the project travels through. Table 3 describes the truncated program values and correlates each value with the Envision criteria that was incorporated into the value. It is important to note how well Envision correlated with the historical values of TVWD and the City of Hillsboro. Both organizations place a high emphasis on being good neighbors and stewards of the environment.

Table 2. Envision Sustainability Credits that make up the Quality of Life Category (Source: ISI 2012)

Quality of Life Category
Subcategory: **Purpose**
• QL 1.1 Improve Quality of Life. • QL 1.2 Stimulate sustainable growth and development • QL 1.3 Develop local skills and capabilities
Subcategory: **Community**
• QL 2.1 Enhance public health and safety • QL 2.2 Minimize noise and vibration • QL 2.3 Minimize light pollution • QL 2.4 Improve community mobility and access • QL 2.5 Encourage alternative modes of transportation • QL 2.6 Improve site accessibility, safety and way finding
Subcategory: **Wellbeing**
• QL 3.1 Preserve historic and cultural resources • QL 3.2 Preserve views and local character • QL 3.3 Enhance public space

Table 3. Truncated WWSP Program Values and corresponding Envision credits.

Credit	**WWSP Program Values**
QL 1.1 Improve community quality of life	Improve quality of life, protect public health and safety
LD 2.2 Improve infrastructure integration **LD3.2** Address conflicting regulations and policies	Engage stakeholders, regulators, communities, and elected officials
LD 1.4 Provide for stakeholder involvement	Foster partnerships through collaboration
RA 3.1 Protect fresh water availability **RA 3.2** Reduce potable water consumption **RA 3.3** Monitor water systems	Seek opportunities for mutual benefits with stakeholders and the public
LD1.3 Foster collaboration and teamwork **LD1.4** Provide for stakeholder involvement **LD2.2** Improve infrastructure integration	Be transparent, offer information to interested parties
QL 2.4 Improve community mobility and access **QL 2.6** Improve site accessibility, safety and way finding	Strive to minimize construction impacts on neighbors – be a good neighbor
LD3.3 Extend useful life **CR 2.3** Prepare for long-term adaptability **CR 2.4** Prepare for short-term hazards	Manage cost and provide high value for ratepayers

Using Envision For Long-Duration Complex Programs

On many large infrastructure projects such as the WWSP, the big decisions are made early in planning. The decision on where to place a large diameter pipeline through a neighborhood or where to locate a 30 million gallon (113,000 m^3) reservoir can have significant impacts on surrounding communities. In the case of WWSP, several communities affected by the program may not necessarily receive direct benefit from the water supply program, so it is even more important to assure the public that the project is being planned with consideration for impacts to the surrounding communities.

In its current form, Envision is well-suited to determining how sustainable a completed project is. Unfortunately, on long-complex projects with schedules that can exceed a decade, third-party verification and final Envision certification does not occur until the project is complete. By that time, the important big decisions have been made and whatever impact was going to happen during construction has happened.

It is possible to use Envision tools to "self-verify" that your project or program is sustainable. However, without third-party verification, there is a risk that the project sponsors (engineers, planners, and owners) will suffer from optimistic exuberance and understate the impact or overstate the benefits of their program on the surrounding communities.

It is easy to see how an engineer could be overly optimistic about a certain criteria and give themselves a score that is higher than can really be substantiated. Once ISI reviewers evaluate the score, the rating is corrected and a more realistic score is given. But if ISI is never involved and the project claims a certain level based on their own optimistic review, then Envision could suffer credibility issues and a project could claim to be more sustainable than it really is, which could hurt the project sponsor's reputation or credibility with the public.

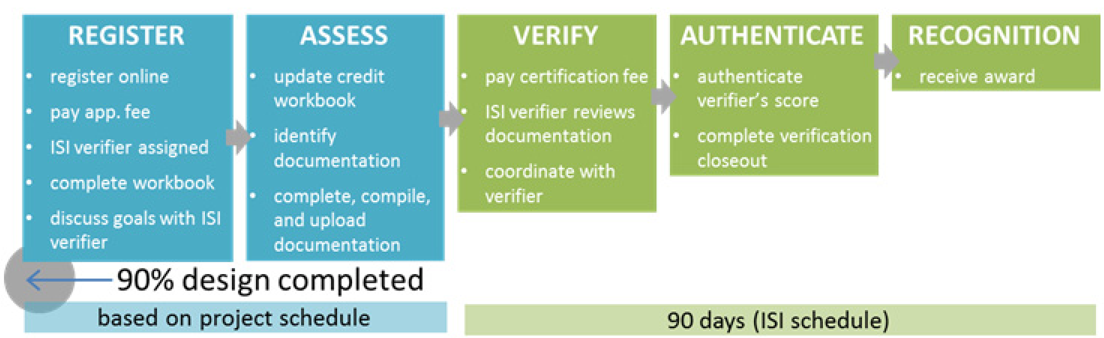

Figure 4. Envision certification timeline for the current Envision rating process – results in Envision certification after final design.

Envision for Planning

An early-Envision process would address some of the challenges associated with applying Envision to long complex programs.

An early-Envision process proposed for use during the planning phase of a program is not a separate Envision rating tool and would not result in an Envision Award. An early-Envision process would enable programs to register the program early during the planning phase and combine the multiple infrastructure components of a program into a single project, using the existing Envision rating system as the framework. For large, long-term, complex programs, adding an extra step to the Envision criteria review process would allow a preliminary assessment of Envision review credits using current rating system to verify that the Program is on the right track to eventually become Envision certified. Similar to the Envision criteria review process, the early-Envision process that is being developed for WWSP will allow the project sponsors to evaluate the WWSP against applicable criteria and then have ISI verify that those criteria were applied correctly.

The WWSP team is working with ISI to develop a first of its kind approach to develop a new process to verify that projects are “Envision-ready” well before they are at a point that they can be Envision certified. Such a process will enable the project team to use Envision to shape the selection criteria for the pipe alignment and reservoir site as well as craft a public message consistent with Envision ideals. The project self-evaluates just like always, but now ISI will be involved verifying the early self evaluation to make sure the project is on track.

This will improve the credibility for the project and for Envision. It will also let the project maximize benefits of Envision by shaping the early frame work of the project and point the project toward sustainability.

How it works

Understandably, ISI is reluctant to certify a project before the project is complete. The new Envision-ready process would not result in an early Envision certification. Instead, the Envision-ready process would formalize an approach or use on large complex programs that are delivered over many years. Projects would not be allowed to claim a certain Envision certification until the project is complete and the Envision criteria have been verified by ISI.

As is done normally in the Envision process, the WWSP has self-evaluated the Program based on the sixty Envision credits. Early in the planning phase it was not possible to fully evaluate all credits. Some were just not applicable at such an early stage of the project.

For example the Quality of Life credit QL 1.3 *Develop local skills and capabilities* does not apply to the WWSP in its current form so the program will not receive points for that credit as part of the early-Envision process.

For those credits that were far enough along to evaluate based on the Envision credit, the WWSP team evaluated the project against the credit and documented the resulting Achievement Level recommendation as illustrated in Figure 6.

QL 1.3 Develop local skills and capabilities

Intent: **Expand** the knowledge, skills and capacity of **the community workforce** to improve their ability to grow and develop.
Metric: The extent to which the project will **improve local employment levels, skills mix and capabilities**.

LEVELS OF ACHIEVEMENT — **NONE**

IMPROVED	ENHANCED	SUPERIOR	CONSERVING	RESTORATIVE
(1) Cost efficient. ☐ The project team proposes significant efforts to hire and train local workers as needed, but mostly hiring specifications directed to the construction contractor. ☐ Programs have articulated goals to meet or exceed industry sector averages. Training is to be done on an as needed basis. ☐ Emphasis placed on hiring and training disadvantaged groups. (A)	**(2) Hire locally.** ☐ The project team lays out broader programs within the project to bring on local firms and workers at higher skill levels. ☐ Local hiring is to extend beyond specifications to the construction contractor and into the project design team. ☐ Training and education is still proposed to be on an as-needed basis. It is not designed to build significant local skills or capabilities. (A)	**(5) Specific skills outreach.** ☐ The project team has developed and committed to affirmative outreach plans and programs to identify and hire local firms and workers at a broad range of skill levels. ☐ Education in some specialty areas will be provided where required. The project team makes an assessment of those educational needs and establishes the requisite education programs. (A)	**(12) Local capacity development.** ☐ The project team commits to working with the community to assess local employment and educational needs. ☐ Specific commitments are made to establish programs to hire and train local workers with an emphasis on minorities and/or other disadvantaged groups. ☐ Plans and commitments for hiring, training and education compared to community needs are proposed. (A)	**(15) Long-term competitiveness.** ☐ The project team commits to working with the local community not only to assess local employment and educational needs, but also to address future community competitiveness. ☐ Working with community leaders, programs are established to identify educational and employment needs and shortfalls. ☐ The team then works with the community to improve and retrofit the local skill base, thereby improving long-term competitiveness. (A, B)

Figure 5. Example of Envision credit that cannot be addressed by WWSP during planning phase. It may be addressed as the program develops in later phases.

QL 1.1 Improve Quality of Life

Intent: Improve the net quality of life of all communities affected by the project and mitigate negative impacts to communities.
Metric: Measures taken to assess community needs and improve quality of life while minimizing negative impacts.

EVALUATION CRITERIA

A	B	C	D
Has the project team identified and taken into account community needs, goals, plans and issues?	Has the project team sought to align the project vision and goals to the needs and goals of the host and affected communities as well as address potential adverse impacts?	To what extent have the affected communities been meaningfully engaged in the project design process?	Has the project owner and the project team designed the project in a way that improves existing community conditions and rehabilitates infrastructure assets?
Yes, Technical Advisory Committee (TAC), stakeholder meetings, meetings with Planning Directors. City Engineers, Public Works Directors, and Elected Officials in each city, PI/PO outreach, future capacity demands, resiliency/disaster preparedness goals, trail incorporation, roadwork piggybacking	Yes, ongoing community engagement and outreach, action/decision/risk log, six public open houses, presentations to City Councils, coordination with City staff, meetings with neighborhood groups. Project developed Program Values that align with Envision to consider needs of communities.	TAC meetings, six public meetings, Regional PI/PO Committee – all affected communities, fire, schools, county, METRO, OSU, others; Water Supply Council consisting of Public Works Directors, City Managers, and Mayors for each community; Website, Virtual public open house, etc.	The project is actively seeking to improve communities located along the route by repaving roadways, building trails, and connecting transportation systems. One of the major goals of the project is to seek opportunities to improve the communities along the corridor. At a minimum, when the project is complete there will be 30+ miles of newly paved roadways stretching from Wilsonville to Highway 26.

Figure 6. Evaluation criteria used to determine Achievement Level of WWSP for Envision credit QL 1.1 Improve Quality of Life.

The WWSP team identified the required documentation needed to support the recommended Achievement Level as show in Figure 7 and identified additional supporting documentation that would be needed before the final Envision rating at the end of the program.

QL 1.1 Improve Quality of Life

Intent: Improve the net quality of life of all communities affected by the project and mitigate negative impacts to communities.
Metric: Measures taken to assess community needs and improve quality of life while minimizing negative impacts.

DOCUMENTATION

Documents IN-HAND for current LOA (CONSERVING)	Documents IN-HAND for potential LOA (RESTORATIVE)
Stakeholder Meetings • Meeting Minutes - public meetings • Meeting Minutes – TAC • Agency Coordination Meetings (minutes and log) Community Leadership Meetings • Meeting minutes with public officials • Letters of support • News or other articles about project support Community Engagement • Reports from public meetings • Documentation of community suggestions • Documentation suggestion evaluation and incorporation when feasible Documents NEEDED for current LOA (CONSERVING) Comprehensive Impact Assessment • Positive and negative community impacts • Mitigation plans • Letters and any published articles (i.e., newspaper or media articles) documenting community support for plan Community Engagement • Documentation of changes to the project to satisfy community needs	Community Improvement/Rehabilitation • Meeting notes documenting understanding of community conditions and assets (notes from TAC meetings, agency meetings) Documents NEEDED for potential LOA (RESTORATIVE) Community Improvement/Rehabilitation • Community endorsement of plans

Figure 7. Documentation needed to support the recommended Achievement Level.

For each Envision credit, the WWSP team also identified changes that could be implemented to move the project to the next Achievement Level in order to earn more Envision credits and a higher final rating.

For each applicable Envision credit, the WWSP team related the individual credit back to the program phases so the project sponsors could see where the credit applies and when specific benefits could be realized by the program.

After the WWSP Team has finished its self-evaluation, ISI reviewers will perform a third-party review to verify that the self-verification performed by WWSP is consistent with how ISI would perform such a review. The goal is to establish a solid realistic baseline for sustainability early in the program.

QL 1.1 Improve Quality of Life

Intent: Improve the net quality of life of all communities affected by the project and mitigate negative impacts to communities.

Metric: Measures taken to assess community needs and improve quality of life while minimizing negative impacts.

RELATED PHASES OF WWSP:

- ***Primary application in Predesign Phase:*** Engage affected communities and stakeholders early in the program to improve quality of life and minimize negative impacts
- ***Primary benefit to Partners:*** Public endorsement and approval
- ***Design/construction phase:*** Manage projects to minimize community impacts

Figure 8. Summary of WWSP Program benefits by program phase related to Envision credit QL 1.1 Improve Quality of Life. A similar table was developed for each Envision rating credit.

CURRENT STATUS AND SUMMARY

The WWSP team and ISI are continuing to develop the Envision-ready process. The name of the process will most-likely change, but the overall approach and outcome should remain consistent. An early Envision process will result in a process that enables large complex programs that occur over many years to use Envision early in the planning phase by applying credits that are relevant or "reviewable" based on the level of planning or design that the program is in. "Reviewable" will include verification by ISI that the program is on-track to obtain a future Envision award.

The benefits of this process will be third-party verification that the project is on the right track; a well-defined framework for sustainability early in the project; and incorporation of Envision values during planning when big decisions such as pipeline routing are made

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Fuchs, J.D., Britch, M.J., Wubbena, T.R., Plattsmier, J.R. 2014, *Planning and Developing a New Multi-Agency Regional Water Supply System*

ADDITIONAL INFORMATION

Tualatin Valley Water District, http://tvwd.org/about-us/awards-and-recognition.aspx

City of Hillsboro, Oregon, http://www.hillsborowatersupply.org/

Willamette Water Supply, http://www.ourreliablewater.org/

Picking a Pipeline Route through a Densely Developed Urban Environment: The Challenges Are Not Technical

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Abstract

To serve the customers of the City of Hillsboro and Tualatin Valley Water District, collectively over 400,000 customers, a new water supply system is in development in western Oregon, south and west of Portland. To select a route and secure a corridor for the critical new facilities, the team developed a holistic approach for evaluating routes that included evaluation of multiple corridors based on criteria developed for technical, social, economic, environmental, constructability, and cost factors. In addition to the evaluation criteria, the approach involved working closely with the partners' public involvement, public outreach and property acquisition consultants to engage affected residents and businesses to help identify appropriate routes and get input. This paper outlines the challenges in identifying pipeline alignments that could be secured to enable the new water supply project be constructed and online by its critical need date of 2026.

INTRODUCTION

To serve the customers of the City of Hillsboro and Tualatin Valley Water District, collectively over 400,000 customers, a new water supply system is in development in western Oregon, south and west of Portland. These two agencies, working cooperatively with several others, are developing the system that will serve to support the growth and economic viability of the area. Additional benefits of the new system include greater redundancy and resiliency, especially seismic resiliency, as the region prepares for a subduction zone earthquake.

The new water supply will take water from the Willamette River through the existing Wilsonville intake facility. A new treatment plant will be constructed near the existing Wilsonville Water Treatment Plant and water will be transmitted from the new treatment plant to a terminal reservoir approximately 20 miles away. Transmission lines will be routed from the terminal reservoir and deliver water to each of the project partners.

In order to move the water from the source at the Willamette River in the southern portion of the project area to the connection points in the northern areas, 35 miles of transmission main ranging in size from 72 to 48 inches in diameter must be constructed. The proposed pipeline route will cross six municipalities – three that will not get service from the project; the local county, federal wildlife refuge areas; the Tualatin River in addition to numerous tributaries, railroads, major highways; and numerous residences and businesses.

To select a route and secure a corridor for the critical new facilities, the team developed a holistic approach for evaluating routes that included evaluation of multiple corridors based on criteria developed for technical, social, economic, environmental, constructability, and cost factors. In addition to the evaluation criteria, the approach involved working closely with the partners' public involvement, public outreach and property acquisition consultants to engage affected residents and businesses to help identify appropriate routes and get input.

This paper outlines challenges in identifying pipeline alignments that could be secured to enable the new water supply project be constructed and online by its critical need date of 2026. Having the system online in 2026 allows the partners to reduce their use of a neighboring water supply before the contract between the two agencies would be renegotiated.

Determining a route for a 72-inch diameter water line through a developed area is surprisingly less of an engineering challenge than a public involvement challenge. Although engineering plays a role in determining which routes are technically feasible and how the pipeline will be constructed, no route is possible without the support, and frequently partnership, of the public and local agencies. It is prudent for project teams to plan ahead to understand how the public feels about the project, determine who may be opposed, which non-governmental organizations will be watching closely, and understand what possible onramps exist for opposition groups to challenge the project. The process described here expands on the process outlined in ASCE M46 as necessary to secure a pipeline alignment in the Portland, Oregon area.

The following sections provide a description of the Willamette Water Supply Program (WWSP) project area and summaries of the processes recommended by the planning and preliminary design team to secure the pipe alignment that will connect the new water supply to its delivery points.

The pipe is long enough and completion deadline far enough in the future that the project partners plan to divide the project into logical construction projects that may proceed independently of adjacent projects. The pipeline will not be built in a linear manner, working from south to north, for example. This influences the routing process by requiring that each option be evaluated for technical feasibility as well as community acceptability. The project can only proceed with the support of the community.

PROJECT AREA

The WWSP is located southwest of Portland, Oregon. The project passes through six municipalities with unique ordinances and bylaws and half of those municipalities will not be served by the WWSP. The pipeline alignment also traverses Washington County between urban growth boundaries and the partners are working with the county to be in the right-of-way of several county roads. Thanks to Oregon's strong land use planning laws, the project areas within city limits are well developed with residences, or public, commercial, or industrial buildings adjacent to public right-of-ways. Outside city limits, the land uses are general agricultural or federal wildlife refuge. The project area is shown on Figure 1. The logical sections for developing route options are: Willamette River Water Treatment Plant at Wilsonville to the SW 124th Avenue project (partnership between the county and partners to place the first three miles of pipe under a new road, from SW 124th Avenue to the finished water storage tanks, and finally from the tanks to the delivery points – one to the east and one to the west.

Other considerations in the project area include:

- Threatened and endangered species
- Cultural and architectural resources
- Wetlands
- Floodplains
- Major utility transmission infrastructure (natural gas, petroleum, electric)
- Seismic resiliency (as a function of soil characteristics and adjacent utilities)

These additional considerations, technical and non-technical, influence routing decisions and are addressed during the routing process.

Establishing a Routing Process

It takes several steps to plan a 30-mile-long pipeline alignment and the mantra for the process should be "there is no perfect route." The routing process allows the routing team to review the project area in increasing levels of detail. The process is illustrated in Figure 2.

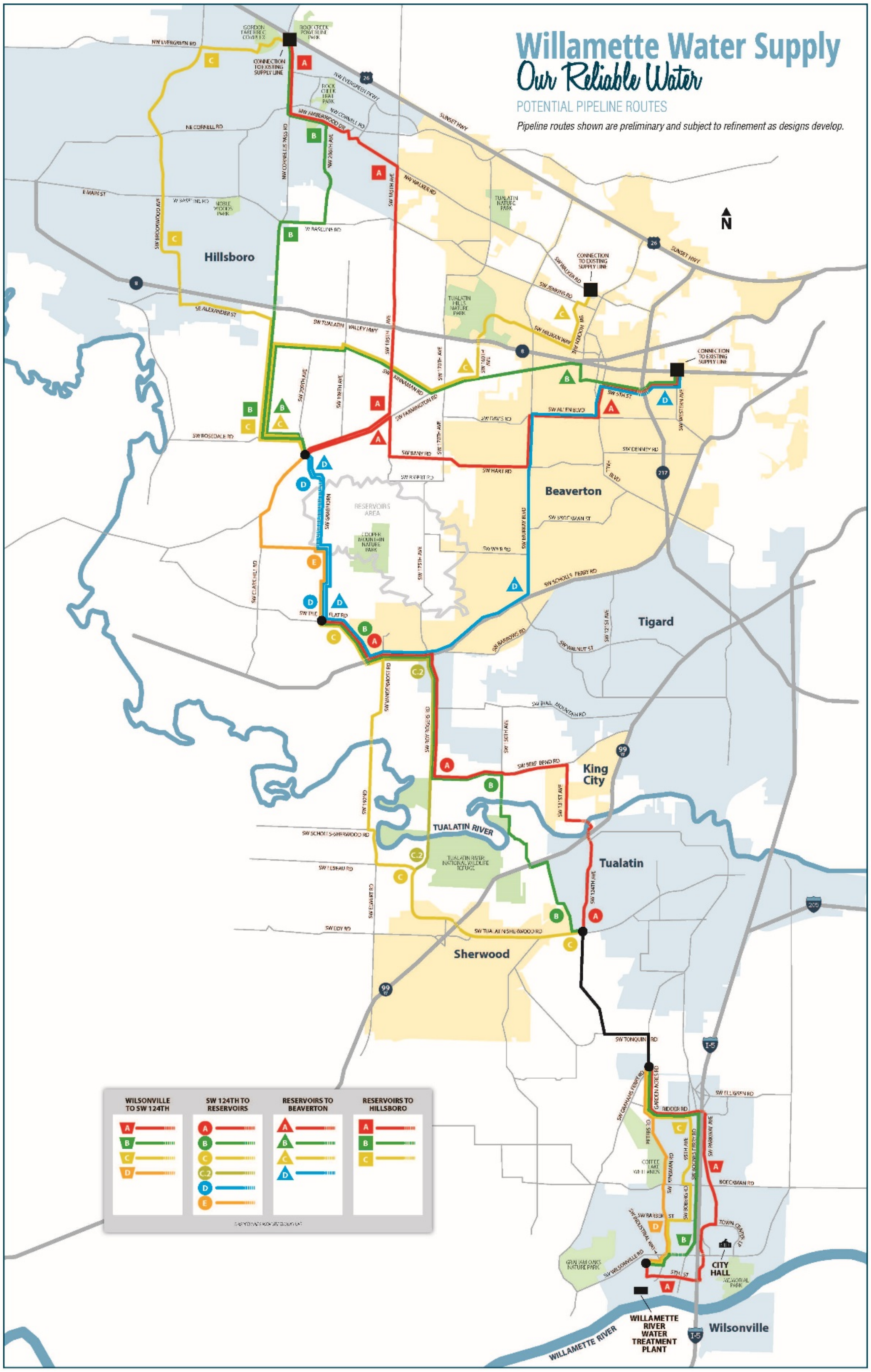

Figure 1. Map of project area showing major project features.

Perform a Desktop Review

The first step was a desktop review to understand the project area and where the opportunities and challenges will be to route the pipe. These challenges can be technical, such as long trenchless crossings of rivers, working around utility infrastructure, or understanding local topography when gathering route options. Trenchless crossings can mitigate some environmental concerns for habitat disturbance, but the costs are much higher than standard trenching techniques. In the WWSP, several major utilities have infrastructure that could pose a risk to the water pipeline if they catastrophically failed during a seismic event. Understanding local topography is necessary to avoid routing options that could require additional pumping. The opportunities noted in the desktop review include known transportation improvement projects or development of trails. With further investigation, it may be possible to partner with those projects to reduce public impacts and possibly reduce costs.

The data collected for the desktop review can also include environmental information including threatened and endangered species zones, critical habitat zones, cultural resources mapping, and known architectural resource locations. All the data collected can be translated into a Geographic Information System (GIS) to display routing considerations on a common map.

Develop a Preliminary List of Viable Routes

Once the desktop review is complete, the route options were refined to develop a preliminary list of viable routes. Project specific criteria (discussed in the following section) were used to evaluate the route options and develop an understanding of their benefits and implementation risks. To evaluate the route options, the best sources of information came from driving the routes and looking for existing major utilities and other potential conflicts that were not noted in the desktop review, and talking with city or other local agency staff who are most familiar with the existing conditions and have knowledge of development and system improvement priorities in their communities. The team recorded observations from the field visits and meeting notes into a criteria matrix that allow the routing team to see the benefits and challenges of each route option side by side. At this point, the team could begin to prioritize or short-list routes that have the least risks and the most opportunities. These prioritized routes are now the technically feasible routes.

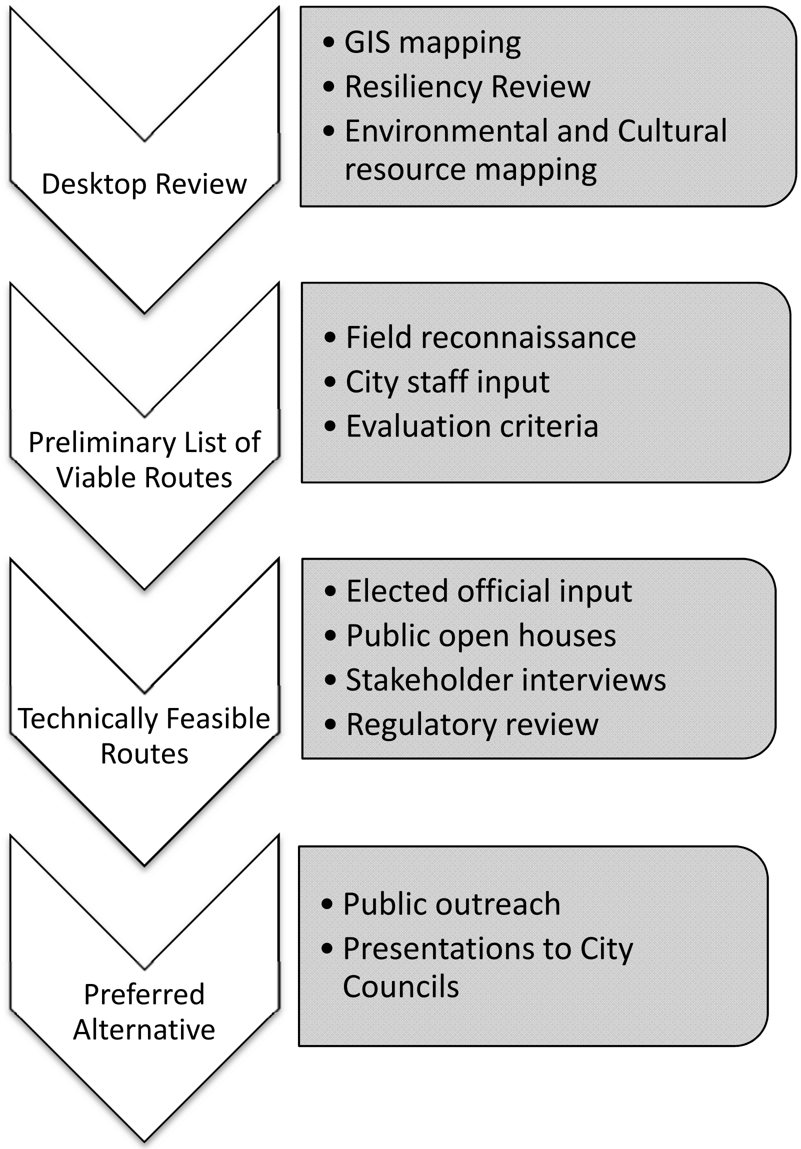

Figure 2.Routing Process

<u>Select Technically Feasible Routes</u>

The technically feasible routes are the routes most likely to be successfully implemented and are the first routes that are ready for public review. Meeting with elected officials of jurisdictions along the route allows them to review the findings and conclusions and provide comments. It is imperative that they understand the purpose and benefits of the program as their support may be needed during the design and construction phases for local permits. Elected officials often know how the public feels about impacts in certain areas of their jurisdiction, which can influence their preferred route. Continue to update the criteria matrix, using categories in Table 1, with best available data to record areas of public concern and highlight areas that are opportunities to partner.

The next two steps proceeded concurrently: public open houses and stakeholder interviews. The team worked with the public involvement and outreach staff to develop and run several open houses along the technically feasible routes. The open houses presented the route options to the public and requested feedback on issues or concerns they have with the routes. More recommendations regarding open houses are included in the following section. The team listened to public feedback and answered as many questions as possible at the open house.

While the open house process is underway, routing staff met with stakeholders to review the data that led to the selection of the technically feasible route. The meetings allowed the team to verify data and work with agency staff to understand transportation and utility projects the jurisdiction has planned. If project timing is aligned, partnering to reduce overall construction impacts benefits the community.

More importantly, if the timing does not align, the jurisdiction's project may become a constraint for one of the technically feasible routes. Several communities in the WWSP area have pavement moratoriums that do not allow utility work to occur in newly paved roads for some period of time, generally 4 or 5 years. Understanding when opportunities become constraints is necessary to move toward the selection of the preferred route alternative.

Continuing to keep the criteria matrix, Table 1, up to date with new information, the team found the criteria matrix to be excellent tool to compare the technically feasible routes side by side and determine which routes have the least implementation challenges.

The final step in the routing process evaluates the technically feasible routes, applies comparative cost estimates to the options, and selects the options with the least implementation challenges or costs. The team clearly documented this final step, discussed it with partner staff, and worked toward a formal adoption of the preferred alternative.

Once the preferred alternative is adopted, the WWSP will reinitiate the open house process to present the route to the public. Additional updates for City Councils are also planned to keep them up to date on decisions and project schedule.

With the selection of the preferred alternative, preliminary design efforts can begin.

EVALUATE ROUTES USING CRITERIA, CONSTRUCTABILITY, COST

Developing the project criteria is an iterative process that combines both the technical and non-technical routing concerns, establishing criteria by which each route will be evaluated. The criteria need to support the potential permitting processes that the project will need to follow. For the WWSP, the expected permitting processes include National Environmental Protection Act (NEPA) and conditional use permits.

The criteria matrices are also useful in discussions with affected jurisdictions or impacted utilities – the completed matrices have all the data necessary to "tell the story" of why a specific route was selected.

The criteria developed for the WWSP have eight main categories:

1. Social/Community Impacts
2. Opportunities/Benefits
3. Environmental Impacts/Permitting
4. System Compatibility
5. System Resiliency
6. Constructability
7. Operation and Maintenance (O&M)
8. Cost

Below each main criteria are several component criteria that are evaluated for each route option, see Table 1. For WWSP, the criteria included rating guidance for three possible scores of "+", "0" or "–": symbols for positive (benefits), neutral (neither benefit or risk), or negative (implementation risks).

Table 1. Criteria for route option evaluations

Criteria/Risk	Definition
Social/Community Impacts	
Congestion/Community Impacts	Number of driveways, traffic volume, major intersections
Impact Critical Facilities	Hospitals, fire stations, emergency services
Community Facilities	Schools, churches, community centers, parks, large employers
Opportunity for Community Enhancement	Add value or benefit to the community
"No-cut" areas	Sensitive community areas that should not be impacted
Opportunities/Benefits	
Proposed Road Projects	"Piggy back"/joint project opportunities
Available Property	Properties currently for sale that provide key sites for staging/tunneling shaft locations
Proposed Development	"Piggy back"/joint project opportunities
Other Project Benefits	Other project benefits and opportunities
Environmental Impacts/Permitting	
Wetland/Waterway Impacts	Amount of jurisdictional wetland/waterway impacted
ESA-listed or Sensitive Species Impacts	Amount of impact to ESA-Listed or sensitive species
Wildlife Refuge Impacts (U.S. Fish and Wildlife)	Cross or impact designated Wildlife Refuge Area
Archeology/Cultural Resources Impacts	Amount of impact to Archeology/Cultural Resources
State Department of Transportation (DOT)	Cross or within DOT right-of-way
Utility (high voltage electrical transmission lines)	Cross or within utility right-of-way
Railroad Crossing	Cross or within railroad right-of-way
County	Cross or within county right-of-way
Community/City	Cross or within city right-of-way
Discharge Locations	Available discharge locations for low point drains and blow-offs
System Compatibility	
Finished water storage tanks	Accessibility and proximity, available right-of-way width
Connection Points	Accessibility and proximity to connection points
System Hydraulics	Compatibility (mostly related to topography)

Criteria/Risk	Definition
System Resiliency	
Geologically Active Areas	Does the alignment cross seismically active areas, liquefaction areas, or Peak Ground Acceleration (PGA) transitions (i.e., rock to silt, etc.)?
High Consequence Foreign Utilities	Are there existing high consequence foreign utilities such as large/high pressure natural gas or petroleum mains, and water transmission mains that would share the alignment?
Transmission Main Accessibility Affected by Seismic Event	In the event of a seismic event will the transmission main be accessible?
Constructability	
Available Right-of-Way	Adequate available right-of-way either existing or associated with an opportunity project
Construction Access	Ability of construction traffic to access work site and deliver materials
Geotechnical	Favorable or unfavorable geotechnical conditions
Utility Conflicts	Conflicts with larger gravity lines, highly congested utility corridors, gas mains
Future Utilities	Planned future utilities that will impact available right-of-way
Traffic Control	Available detour routes and right-of-way width
O&M	
Access	Ability of O&M to access and maintain facilities
Future Right-of-Way Changes	Future right-of-way changes will affect access to the transmission main
Cost	
Capital Cost	Cost to construct
O&M Cost	Life-cycle cost of O&M

TELL THE STORY: PUBLIC INVOLVEMENT AND OUTREACH

Public involvement activities set the stage for positive public interactions and are likely the first people the interested public will see as the "face" of the project. Multimedia communications are necessary to connect with the public and the options grow wider each year. Competition for the public's attention is staggering. Project communications need to be professional, vivid, and interesting to get noticed. Standard notices can be sent through the mail to notify neighbors about upcoming open houses. Other recommendations include:

- Develop focus groups to test run messaging (FlashAlert Newswire 2014)
- Flyers in utility billing statements (paper or email)
- Project website that can be a virtual open house
- Social media updates (only if the project is committed to developing timely, interesting sound bites and photos to post)

Open houses can provide a venue to share the project with the public and, in projects of this magnitude, two sets of open houses are recommended. Present the technically feasible routes, discuss the criteria used to develop them, and collect feedback. Once the preferred alternative is adopted, the WWSP will repeat the open house process and to talk about construction timing and traffic impacts.

Updates to elected officials of all levels were created to keep the purpose and importance of the project forefront in decision maker minds. Updates and briefings were timely and focused to present the essential information necessary to answer general public questions should they arise. No one likes to be surprised by a megaproject in their jurisdiction they were unaware of, or receive questions for which they are unprepared. These updates help keep all levels of government aware of the project progress.

APPROACH PROPERTY ACQUISITION WITH CARE

Acquiring private property is challenging for utilities project. When properties are needed to secure an alignment, the project team tried to understand the following:

- How critical is the property to the preferred alternative?
- Is the landowner willing to sell?
- Have there been challenges to similar land acquisitions in the community?
- Does the community have any needs (i.e., parks, trails, community gardens) that can be filled by the project?

Property acquisition is vital to securing the WWSP pipeline corridor and sending the right team out is critical. The team needs to understand the communities, project history, previous challenges, and be able to make a case for the future use of the property. Use of eminent domain (condemnation) is possible, but for the WWSP, the project will only work with willing sellers at this stage in project development.

COORDINATE FREQUENTLY WITH LOCAL AGENCIES

The team provided proactive agency coordination at all steps of the planning process keeping local jurisdictions aware and up-to-date with project progress. For the WWSP, focused meetings with technical staff occurred during the preparation of the viable routes and again when the technically feasible routes were selected. Depending on the relationships already established between project staff and agency staff, additional meetings were worthwhile to build understanding of the project purpose and need as well as gaining awareness of the local jurisdiction's needs (e.g., areas to avoid, upcoming transportation or utility improvements). The local agency staff can become project allies and help with future permits and coordination to benefit the project. The Program is developing memorandums of understanding (Kulla 2014) and eventually intergovernmental agreements with local agencies to begin coordination with clarity of signee responsibilities.

CURRENT STATUS AND SUMMARY

This process is the one developed and used on the WWSP to select a route and secure a corridor for the critical new facilities. The holistic approach to evaluating routes included evaluation of multiple corridors based on criteria developed for technical, social, economic, environmental, constructability, and cost factors.

In late 2014, the project team completed the steps necessary to make a recommendation of the preferred transmission pipeline alignment. To make the recommendation, the project team has developed the criteria that were used to evaluate potential pipeline routes. The public involvement and public affairs consultants have been integrating with the engineer, planning, and permitting teams and at public open houses to present the shortlisted routes to the community.

Preferred route selection is scheduled to be approved in early 2015. Throughout 2014, the permitting team met with agencies and interested parties to develop a permitting strategy. In 2015, preliminary design will begin on the preferred pipeline route, reservoir, and pump station. The project is well on its way to begin design and come online in 2026.

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Additional Information

Project website: http://www.ourreliablewater.org/

DC Water Uses 3D FEM in Assessing Century Old Trunk Sewer

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Abstract

In a recent construction, a new four story building was constructed over a 110 year old brick arch sewer (22' wide and 23'6" high). DC Water learned of the construction after it was completed when the developer applied for water and sewage connection. The developer submitted a structural engineering assessment in an effort to prove that the building does not compromise the brick arch sewer tunnel. Developer's engineer used a simple load-for-load comparison that computes vertical loads only to substantiate that the load of the wood framed building including excavation for basement did not exceed the previous earth load on the sewer. DC Water engineer rejected this simple arithmetic computation as the calculation failed to analyze the impact of the unbalanced load on the masonry arch sewer. Fully aware of the potential of its impact, DC Water's engineers responded by immediately organizing a series of engineering evaluations that consist of in-house FEM (finite element model) structural analysis as well as a geo-structural FEM by a consulting engineer. In this paper, we discuss the results from the two independent FEM analyses. Both these analyses confirm a similar stress/strain mode (tension) on the brick arch sewer. Armed with the results of the analysis, DC Water engineers focused on the tension zones during the inspection and identified a crown fracture under the new building. This paper will provide details and owner's lesson learned regarding the impact of the new construction on a conventional masonry tunnel with modern day engineering tools.

1. BACKGROUND ON THE TRUNK SEWER

DC Water has a combined sewage collection system with a few large tunnels built more than a century ago. The NorthEast Boundary Trunk Sewer (NEBTS) is one of the largest sewer tunnels (22’ wide and 23’6” high). This sewer was originally built in 1905 with a peak capacity of 2,200 MGD. As per the current model, this sewer transports up to 3,500 MGD in a 15-year rain event, with a 5-ft surge. This red brick masonry sewer remains in decent shape after 110 years in continuous service. However, its integrity became questionable when a developer built a four story residential house on top of the sewer. DC Water learned of the construction late, when the building was ready and the developer applied for water and sewage connection.

The impacted segment has a “mushroom” shape as shown in Figure 1. The sewer is entirely made of red bricks above the spring line. The crown area is made of five courses of red brick. The base of the sewer tunnel is on concrete masonry lined with two courses of red brick. As shown in the Figure 1, the lower base of the sewer tunnel appears sturdy even with minimal steel rail reinforcement.

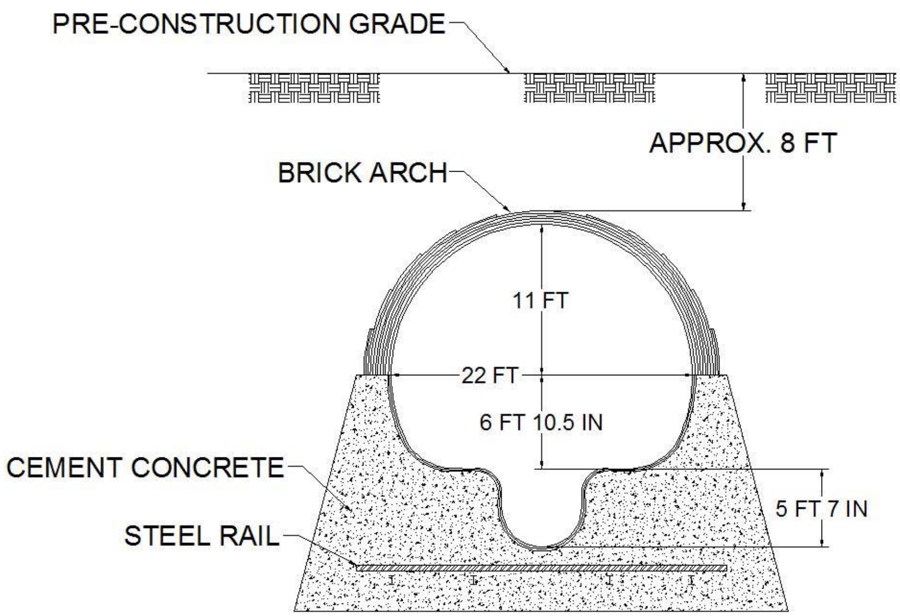

Figure 1 - Sewer tunnel section under the unauthorized building

The new building structure consists of the 4 story wood framed building with brick veneer. The building has a partial basement and is almost directly above the sewer but the orientation is about 30 degree skew. See Figure 2, with the site plan for the orientation of the building with respect to the sewer tunnel.

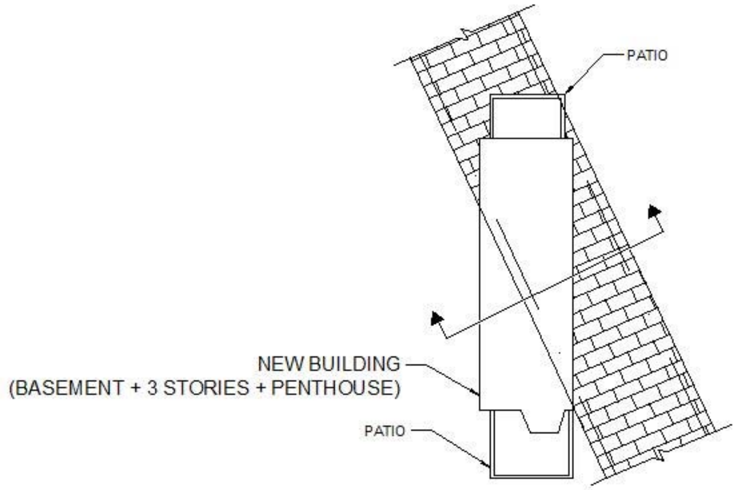

Figure 2 - Layout of the new construction over the existing sewer

Section cuts perpendicular to the sewer tunnel demonstrates how the building exerts a varying non-symmetric load to the sewer tunnel within the zone of influence. Developer's engineer ignored the orientation of the building and submitted assessment assuming a symmetric loading as shown in Figure 3. Basement construction impacts earth pressures on the brick arch as per the layout, but was not considered by the developer's engineer.

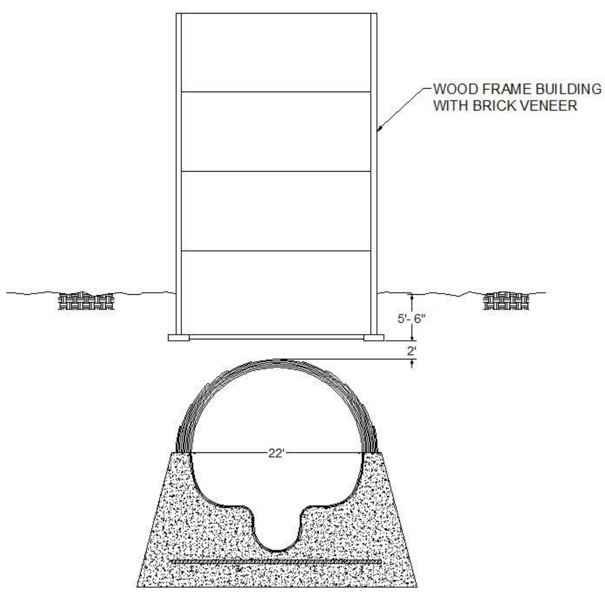

Figure 3 - Impact assessment by the developer

DC WATER engineer rejected the developer's impact assessment as it did not consider the true loading conditions. DC WATER review comment as shown in Figure 4, explains that the evaluation should consider the unbalanced loading.

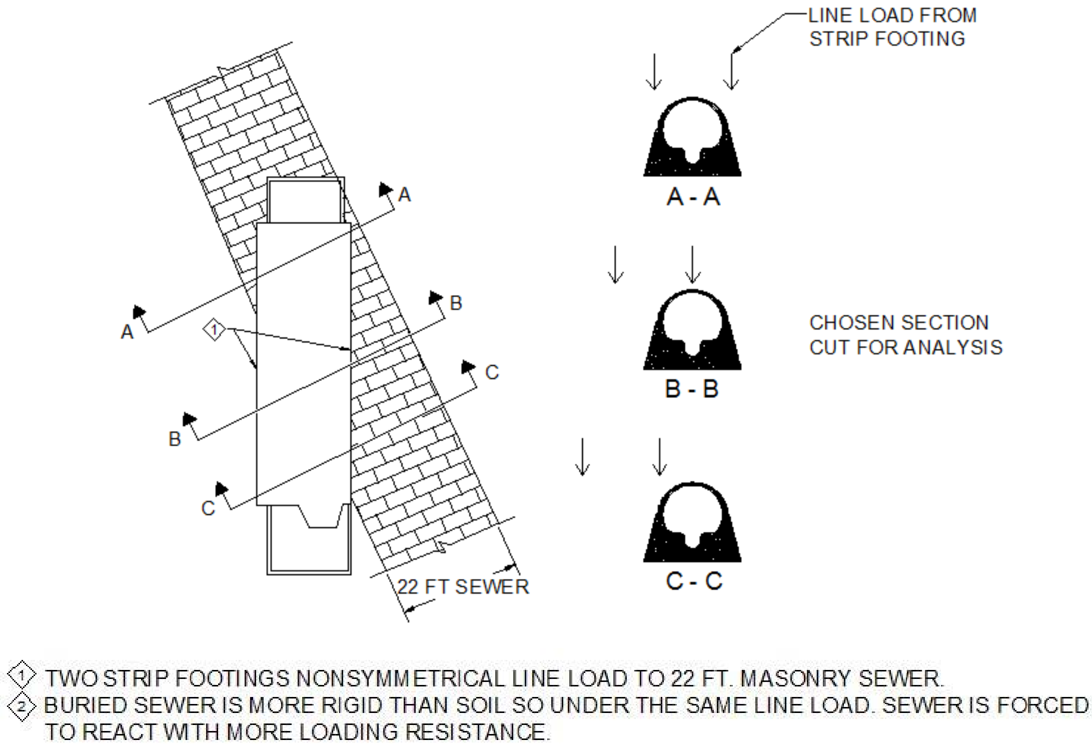

Figure 4 - Illustration of the flaws in the developer's assessment

The masonry arch makes it more susceptible to unbalanced load, as brick masonry cannot resist any tension. Due to the complexity of the geometry, DC Water decided that a 3D analysis will be required to analyze the response of the structure to the loads.

2. DC WATER FEM ANALYSIS

To investigate the impact of the unbalanced loads, DC Water's Finite Element Method (FEM) of analysis was used with a 3D model of the sewer. A 100 feet length of the sewer structure was modeled with building loads at about 30 degree skew with respect to the center line of the sewer as shown in Figure 2.

The new structure is a wood framed building and the most significant loading on the sewer is the unbalanced unloading of the earth due to the basement construction. The model was developed to capture the impact due to the change in the loads on the structure including earth pressure.

There were several phased loading conditions related to the new residential building and the level of CSO in the sewer tunnel. To investigate the impact of the unbalanced loads, DC Water's FEM analysis included fluid pressures for empty (sewage flow in the cunette only), partial full (normal rain event with CSO up to spring line of arch) and for 5ft surcharge (15 year storm event).

Pre-construction loading is shown in Figure 5. Before the construction of the sewer, the design loads along the sewer were predominately uniform from lateral earth pressure at rest and the weight of the soil above the crown.

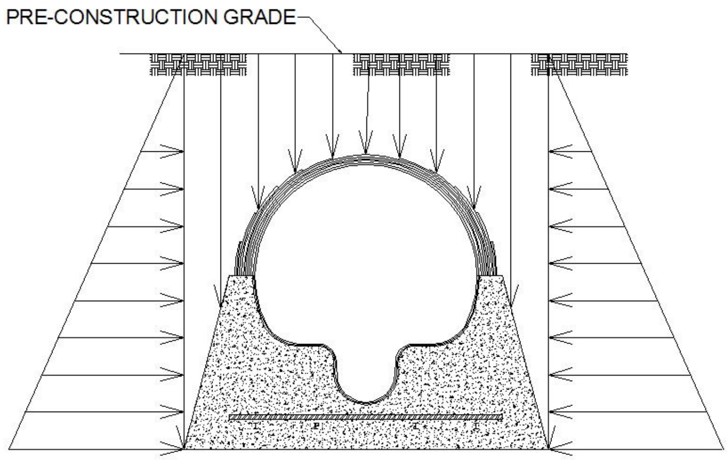

Figure 5 – PRE-Construction design loads

Post construction loading is shown in Figure 6. Earth loads shown in Figure 5 are further modified to reflect the unloading of the weight of the soil removed for the partial basement excavation and reduced lateral earth pressure based on the footprint of the building.

In addition to the earth loads shown below, the building load, such as line loads based on the typical floor framing, was also included in the FEM analysis.

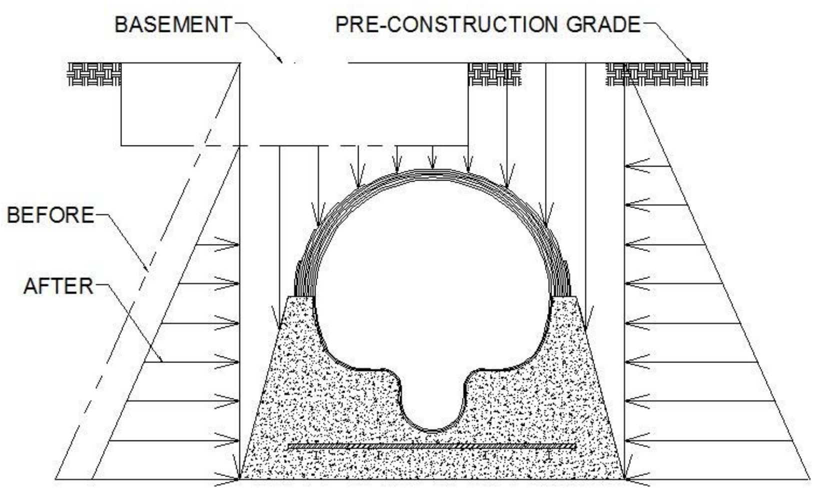

Figure 6 – POST-Construction design loads

Although Figures 5 and 6 show typical 2D sections, but as the building is in a skew, a 3D model was used to capture the impact of the layout for the loads on the existing sewer structure.

3. FEM ANALYSIS – StaadPro and PLAXIS

The analysis was done in both StaadPro and PLAXIS. Both the models used 3D modeling to determine the impact of the new construction based on the alignment of the sewer and building.

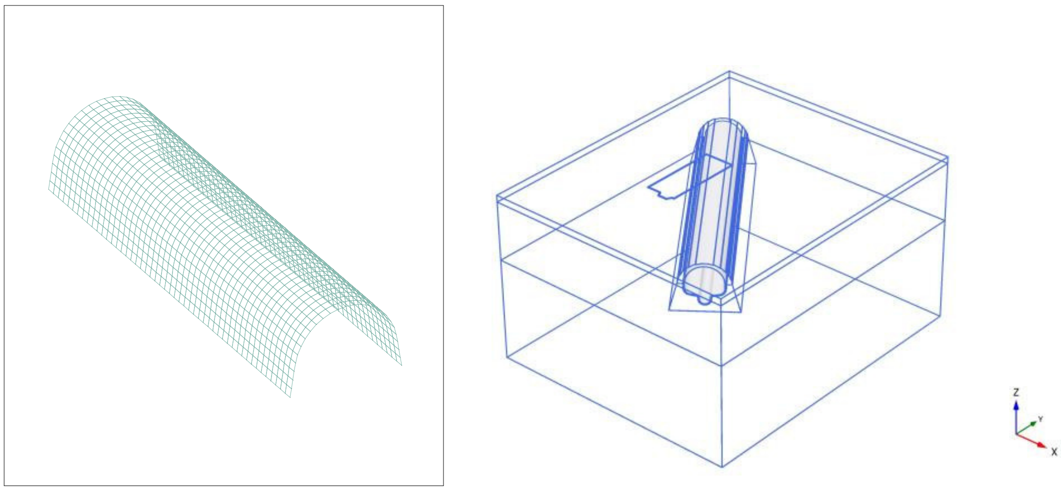

Figure 7 - StaadPro model (left) & PLAXIS model (right)

As it is difficult to determine the engineering properties of the century old construction and backfill, the analysis focused on evaluating the impact of the loads on the existing sewer for pre-construction and post-construction loads. Comparison of the results of the pre-construction and post –construction loads with the same assumptions regarding the soil and material properties was used to determine the impact of the new construction.

Most critical stresses on the structure were in the hoop direction due to Axial force (Sx) and Moment (Mx). The stresses in the brick arch were further evaluated for the stresses on the inside face (bottom of plate element) and outside face (top of plate element) of the FEM model of the structure.

Bottom combined stress (inside face) for pre and post construction loading is presented in Figures 8 and 9 respectively.

Stress diagram shown is for the stress in the plate element in the inner face of the sewer computed with Axial force (P), unit area (A), Moment (M) and Section modulus (Z) for the plate element as computed by STAAD Pro.

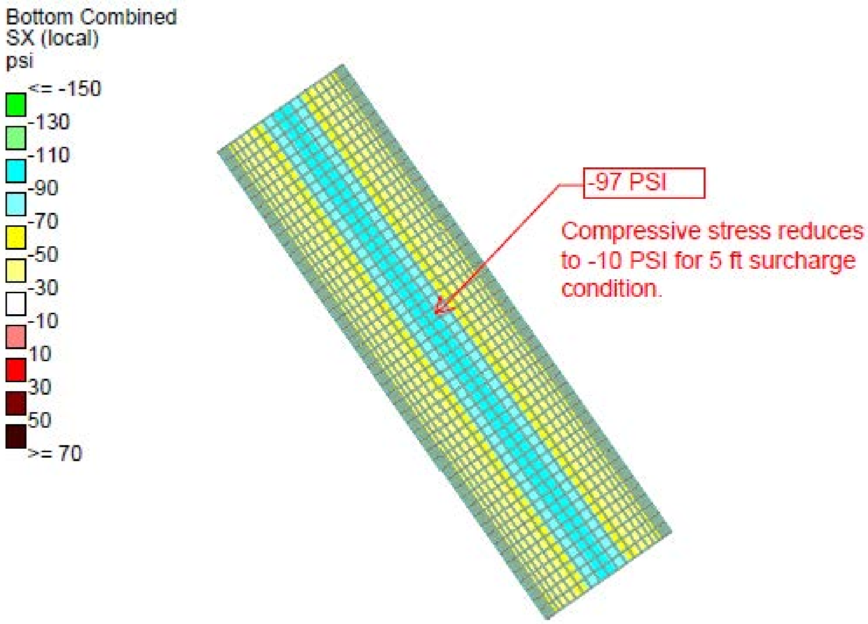

Figure 8 - Combined hoop stress (P/A + M/Z)- Inside face of arch PRE-Construction loads

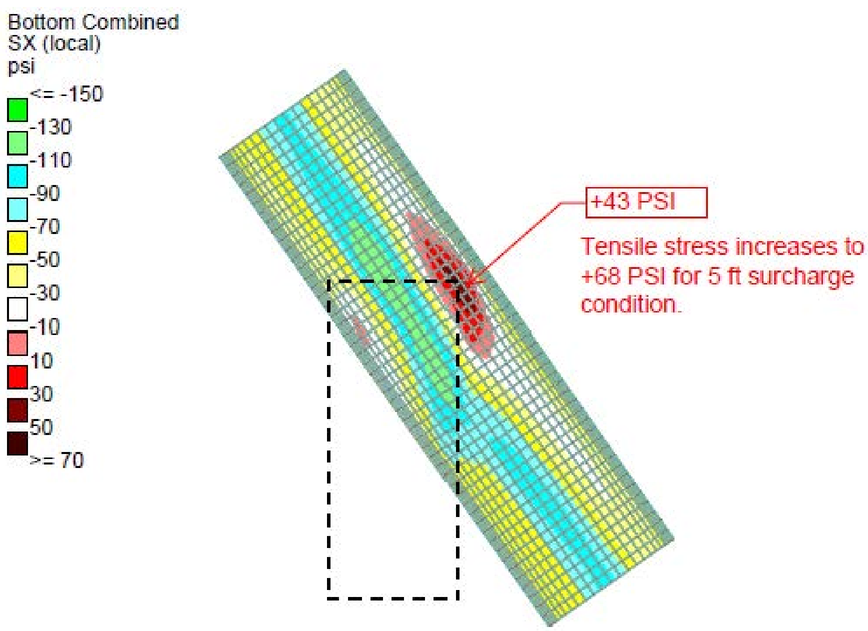

Figure 9 - Combined hoop stress (P/A + M/Z) - Inside face of arch POST-Construction design loads

The field inspection, with the knowledge of potential tension zones in the structure focused on these areas during field survey and were able to identify cracks in the crown very close the tension zones in the FEM model. Survey results in the tension zones are presented in a subsequent section.

The Top Combined stress (outside face) of the structure was also evaluated for both the tension zone and is presented in the Figures 10 and 11. The analysis also demonstrates tension zone on the outside face but this could not be observed for a buried structure.

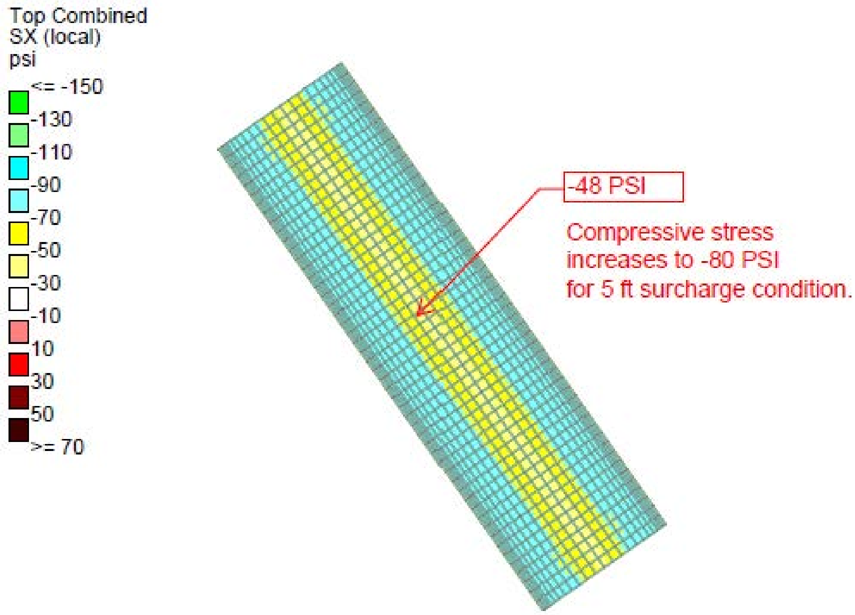

Figure 10 - Combined hoop stress (P/A + M/Z) - Outside face of arch PRE-Construction design loads

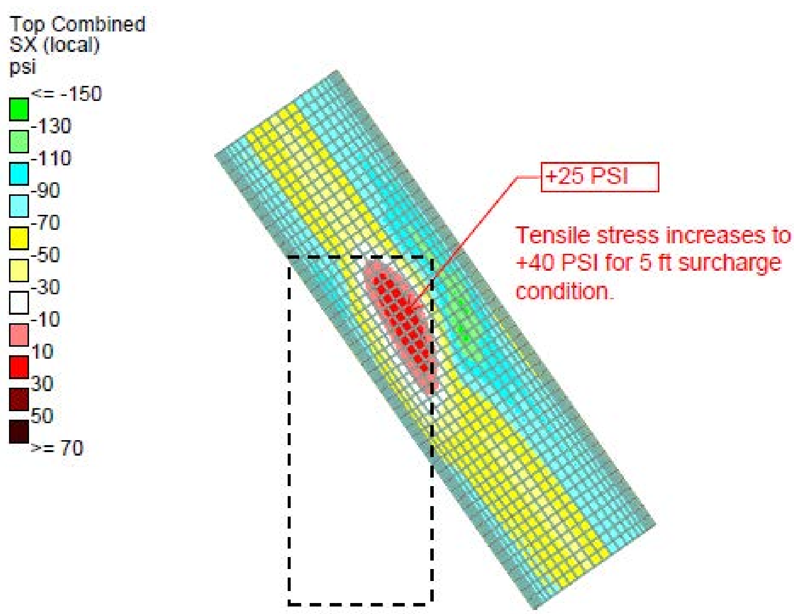

Figure 11 - Combined hoop stress (P/A + M/Z) - Outside face of arch POST-Construction design loads

Only two load cases are presented in this paper, however analysis further evaluated the structure for the loads due to water pressure inside the pipes. Most severe loading was internal pressure during the 15-year storm as estimated by the hydraulic analysis. These are shown in the notes on the above screen shots.

As can be seen from the pre-construction and post-construction load combinations, the brick arch of the structure is in tension in the post-construction load case only.

STAAD results were also compared with PLAXIS results by the consultant working for DC Water. The deformation (strain) of the existing brick arch are similar to the STAAD analysis. See Figure 12 for PLAXIS deflections.

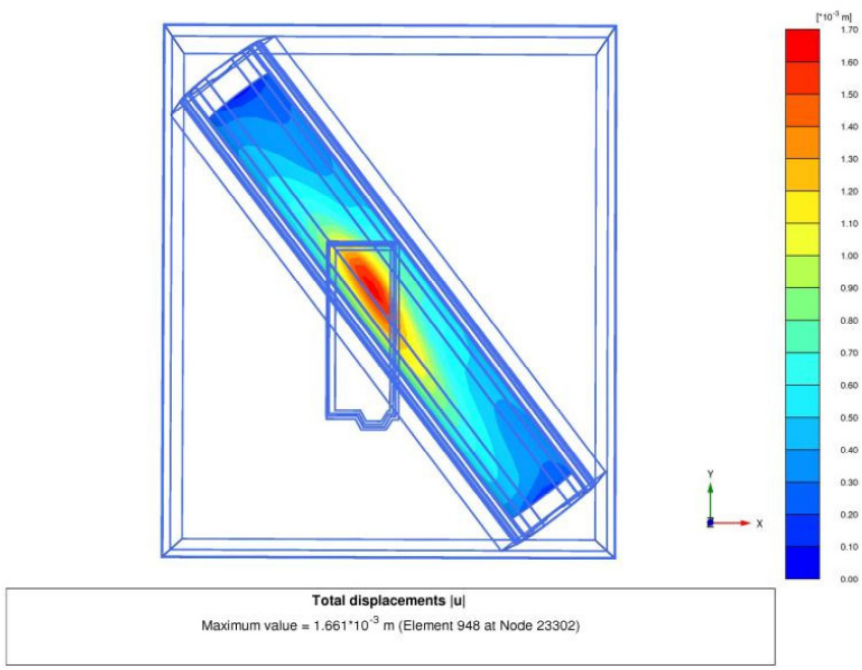

Figure 12 - Deflections as per PLAXIS model.

4. INSPECTION

DC WATER engineer entered the sewer tunnel to scan the crown beneath the unauthorized building structure armed with the concerns identifying the tension zones indicated by the FEM analysis of the structure. During inspection, we found a ¼" wide longitudinal fracture of 15-ft length within the zone of influence (within the building footprint) of the new building structure.

DC Water set up a monthly entry inspection schedule to monitor the fracture after it was first identified on November 22, 2014. We anticipated the fracture will likely progress within the masonry arch to search for a renewed post-fracture balance.

The propagation of the fracture has exceeded our expectation in terms of speed and extent. By the time of completing the scaffolding to deploy a 100-ft CFRP repair in February 19, 2015, the original fracture expanded to 145-ft length and two more localized longitudinal fractures were observed. See typical crack picture in Figure 13.

Figure: 13 - Crack at crown (+/-12 O Clock position)

The two FEM analysis achieved their objective in predicting the damage mode, i.e., tensile fracture zone. Field inspection result confirmed the serious nature of the problem identified with the in-house by DC Water's FEM analyze.

5. LIMITATION OF THE LINEAR ELASTIC FEM ANALYSIS

The progress of the fracture could not be projected by our FEM analysis, as the FEM tools we deployed are effective only in predicting the tension zones with a linear, elastic analysis only. The analysis only determines the tension zone in the structure, and when the structure cannot take any tension (brick masonry), the failure mode cannot be predicted with the linear analysis.

Once a crack develops locally, it is likely that the tension zone expands longitudinally to redistribute the loads. A mode of progressive failure would demand a more-sophisticated FEM tool and a defined geotechnical boundary condition including modeling of the cracks.

6. REPAIR

The rapid progress of the crack, lead to development of an emergency repair to one of the most critical structure in the sewage system. DC Water engaged a design builder to engineer a repair with CFRP to incorporate multiple existing fractures. This is expected to be a nonlinear non-elastic FEM design incorporating the known cracks, as we do not believe the previous elastic linear FEM analysis could predict the performance in post-rehabilitated stage of a fractured masonry tunnel.

DC Water allowed a composite design using the compressive capacity of the host masonry tunnel in the FEM model for the design of the CFRP repair. We did not adopt a “fully deteriorated” design approach. The repair design accounted for the existing masonry arch tunnel to not resist any significant tensile stresses in structure due to the loads.

The repair design also includes the loads due to demolition of the existing structure so that after the emergency repairs, the unbalanced loads due to the new building can be removed and the structure stabilized.

7. LESSON LEARNED

- Masonry sewer tunnel tends to be older in the collection system, built before the modern “soil mechanics” established in 1930’s.
- Zone of influence of a buried masonry sewer arch is under appreciated within the professional structural engineer community. Modern day masonry design is typically not used for arch structures.
- Masonry sewer tunnel assessment demands geo-structural expertise that evaluates the impact of the potential of unbalanced load. A load-for-load comparison that may be acceptable for above grade reinforced structure is not valid here.
- The confirmed new live fracture warrants a quick action by owner as the masonry sewer tunnel may encounter more fractures before it can be stabilized for geo-structural loads triggered by the unbalanced load.
- Incorporate the fracture(s) and unbalanced load condition in the design of retrofit. Typical pipe design does not account for unbalance loads and FEM analysis considering soil structure interaction is required.

Wilburton Sewer Improvements—No Problems, Just Opportunities to Provide a Toolbox of Engineering Solutions

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Abstract

The City of Bellevue serves as a major financial hub in Washington state, and continues to experience high growth in its downtown and adjacent areas. The Wilburton sewer basin lies just east of downtown Bellevue across Interstate 405 (I-405), and its existing sewer system originally built in the 1960's needs capacity improvements to support the anticipated growth in this area. After an initial condition assessment and pipeline alignment alternatives analysis, the existing sewer alignment was selected as the preferred route. However, after 50 years of development, this required designing 4,300 feet of new 12-inch to 30-inch diameter sewer through a peat bog and creek, adjacent to a new 30 feet tall retaining wall holding up I-405, across 10 lanes of I-405, through a Lexus car dealership, and under a major arterial. These various and distinct challenges required a full range of project solutions including auger-cast pile supported pipe, geofoam backfill, prescriptive shoring methods, trenchless techniques, complex bypass and construction sequencing, and mitigation of business impacts. This paper summarizes the key elements of the project; the alignment evaluation process; the numerous challenges and constraints along the selected alignment; and the toolbox of engineering solutions required to provide a constructible new pipeline for the City of Bellevue. Design was completed in April 2015, and construction is scheduled to begin in the summer of 2015.

Keywords: Pipeline design; Trenchless technologies; Constructability.

Project Overview

The purpose of the Wilburton Sewer Capacity Improvements project is to replace sewers in the Wilburton sewer basin area to accommodate projected future higher density, mixed-use redevelopment. The Wilburton service area is located along the I-405 corridor from NE 8th Street south to SE 8th Street (Figure 1).

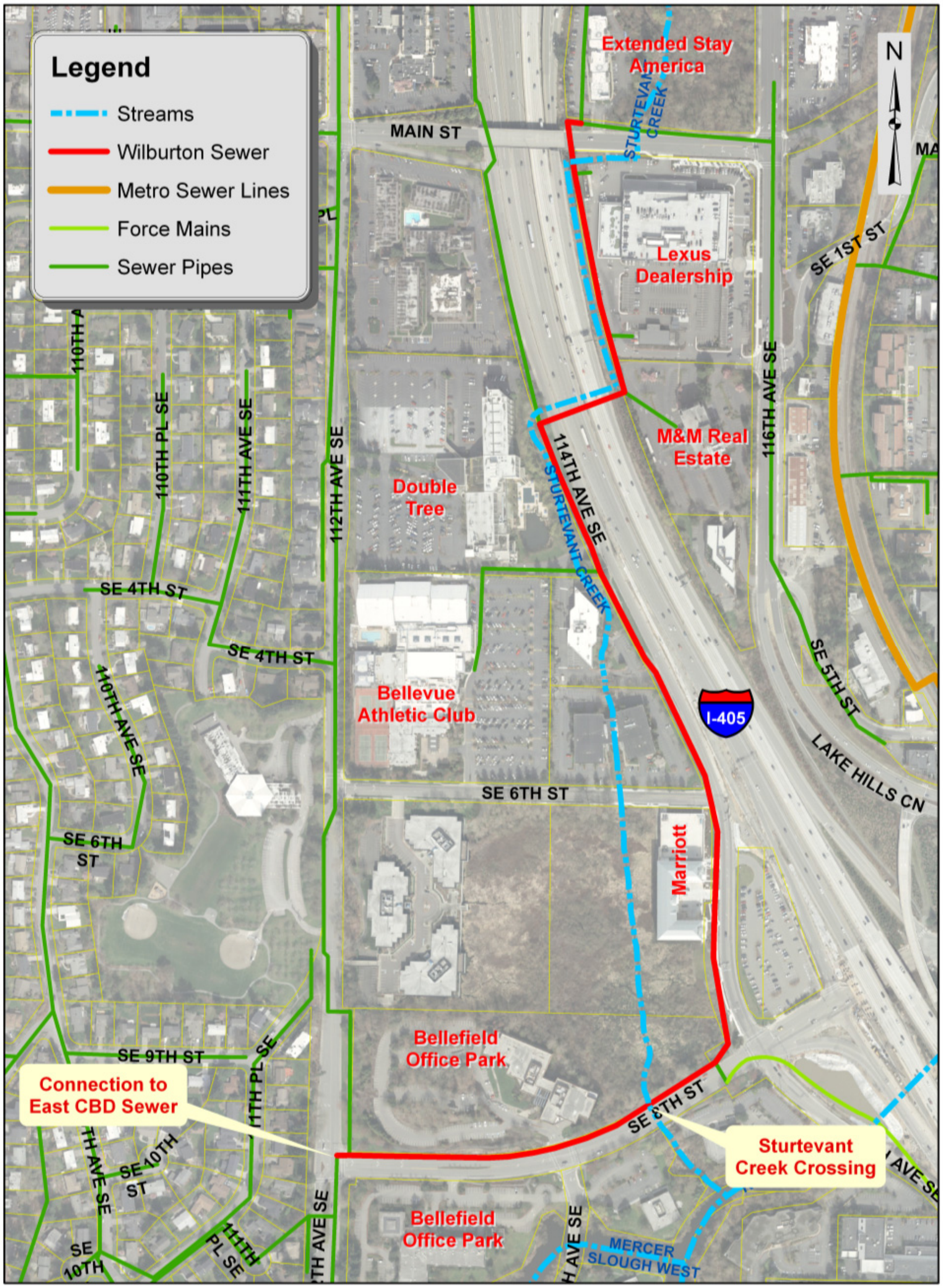

Figure 1. Wilburton Sewer Site map

The project upsizes approximately 4,300 lineal feet of sewer to allow increased flow from the sewer basin (projected to be 4.18 mgd at 2064 "buildout"), and discharges into the City of Bellevue's East Central Business District Trunkline and the Bellefield Pump Station – two additional projects currently being constructed to accommodate increased flows from downtown Bellevue and the Wilburton basin.

Alternate Alignments. During the predesign phase of the project, sewer alignment alternatives were considered in addition to following the existing alignment. One option considered routing the new sewer across I-405 with a new trenchless crossing north of Main Street and then south along 114th Ave SE west of I-405. This would have avoided impacts through a car dealership on the east side of I-405 where the existing sewer was located. Another alignment considered was to route the new sewer along SE 6th Street rather than SE 8th Street since the ground conditions were much better along SE 6th Street; however, this would have required a 40 feet deep sewer at the west end, which was 32 feet deeper than the existing sewer trunkline connection. A final major option considered was to route the new sewer either within SE 8th Street or on the south side of SE 8th Street (on private property); however, these options resulted in greater impacts to the roadway that was recently reconstructed with a complex geofoam pavement section, or greater impacts to private property and its tenants as compared to being on the north side of SE 8th Street where the existing sewer was. Ultimately, after reviewing other alignment options, following the existing sewer alignment was selected as it resulted in the least impacts and allowed existing service connections to be connected more easily to the new sewer.

Constraints and Challenges

Poor soils and creek/wetland crossing along SE 8th Street. The project has several different types of unique challenges that required different design solutions along the 4,300 feet alignment. At the downstream end of the alignment along SE 8th Street, the sewer is located in very soft, compressible peat which overlies lacustrine and alluvial deposits, ranging in depths of up to 50 feet (Figure 2). Due to these poor soils, SE 8th Street in this area had experienced settlement in excess of 5 feet, and was reconstructed with a complex geofoam pavement section that minimized settlement of the historically "sinking" roadway. The alignment also crosses Sturtevant Creek and adjacent wetlands that feed into the Mercer Slough, a highly environmentally sensitive natural and recreational resource in south Bellevue. Any sewer designed along SE 8th Street had to provide sufficient support for the pipeline, yet also minimize impacts to the roadway and creek/wetlands.

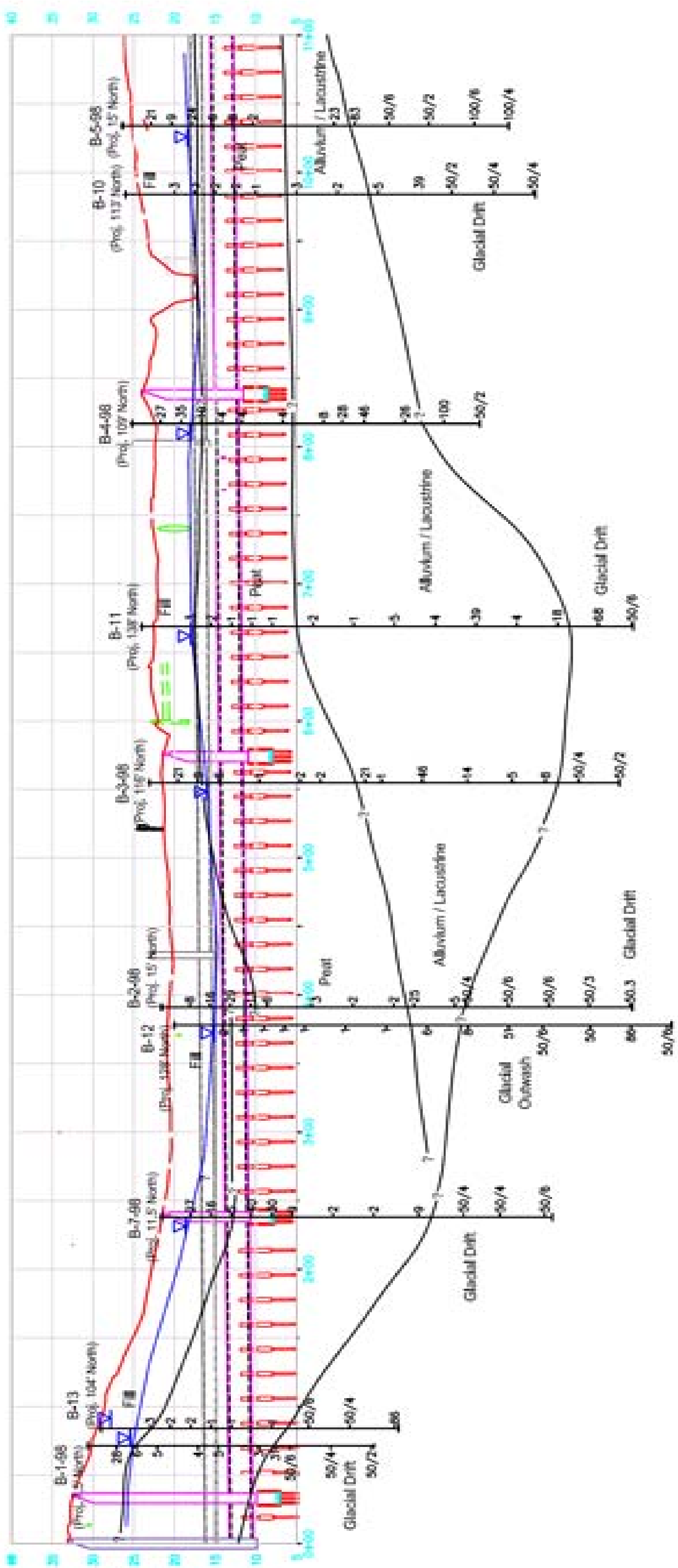

Figure 2. Geologic profile along SE 8th Street

In order to provide support of the 30-inch diameter pipeline above the compressible peat, pipe support options were evaluated including auger-cast piles, steel piles, and prestressed concrete piles. Ultimately, auger-cast piles with pre-cast pile caps and saddles were selected due to less noise and vibrations impacts, most corrosion resistance, least cost, and ability to adjust to field conditions. Key elements of the design included:

- 30” ductile iron pipe, Class 56 (0.63” wall thickness) to maximize the pipe section modulus (i.e., maximizes allowable spacing between piles)
- 18-inch diameter piles @ 18 feet spacing that matches DIP pipe segments
- minimum 12 feet pile embedment into the bearing layer (Figure 2)
- pre-cast pile caps and pipe saddles that saves construction schedule and allows field adjustment to final pipe invert
- maximum of 8.5 feet of fill soil above the ductile iron pipe due to structural limits of the pipe section (Figure 3)
- geofoam backfill above pipe to minimize loads on DIP
- minimum soil cover above geofoam of 2 feet to counteract buoyancy (Fig. 3).

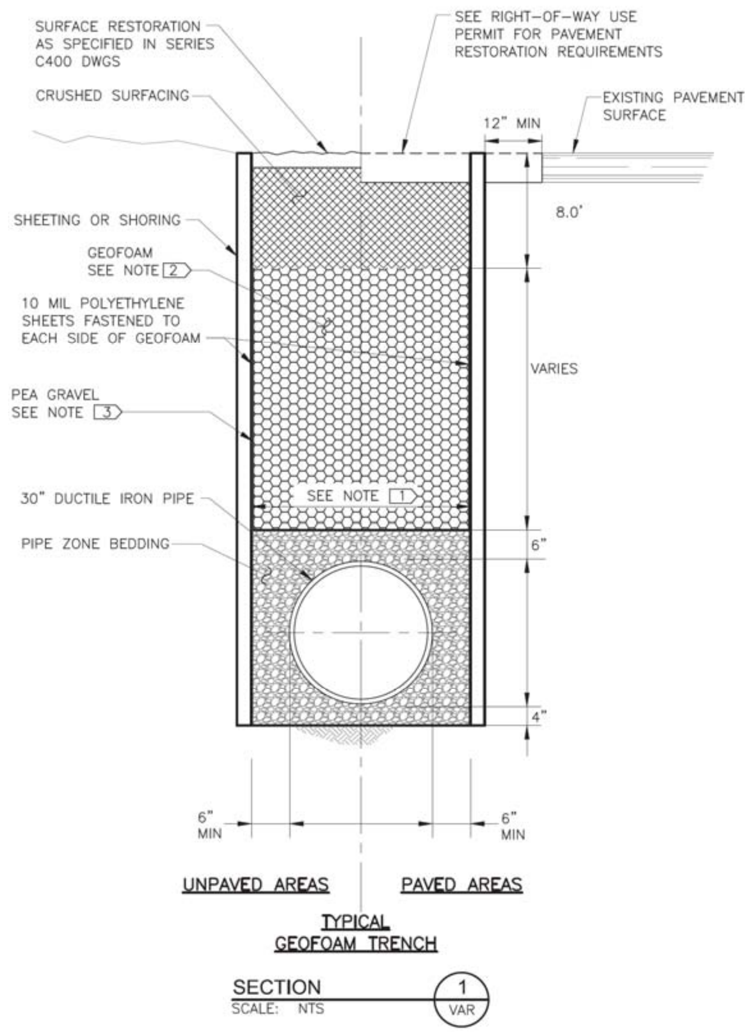

Figure 3. Geofoam trench backfill section

For the creek and wetland crossing, both trenchless and open-cut methods were evaluated to determine the most cost-effective way to get across the creek, minimize impacts to wetlands, and ability to obtain permits from various agencies. Due to the groundwater conditions at the creek, various trenchless methods were ruled out quickly including auger-boring and open-shield pipejacking. Microtunneling was also ruled out due to the soft peat and prohibitive cost for the relatively short crossing (approx. 50 feet). Guided pipe ramming appeared to be the most feasible trenchless method that would provide grade control, handle the groundwater, and be a relatively cost-effective method for the short crossing. However, after further evaluation, this method would require a minimum 5 feet depth of cover below the creek bottom to mitigate for the risk of creek water flowing into the steel casing during the pipe ramming. This 5 feet depth requirement would then require the new sewer and new downstream pump station to increase in depth accordingly, and thus significantly increase project costs. Therefore, a decision was made to open-cut across the creek that would allow 2 feet of cover depth over the sewer, and minimize downstream impacts to both the new sewer system and pump station being designed.

Key elements required for the open-cut creek crossing included:

- hydraulic/scour analysis was completed to confirm 2 feet of cover below creek bottom was sufficient. This was reviewed and approved by one of the local permitting agencies (Washington State Dept. of Fish and Wildlife)
- temporary dam and pumped creek bypass, including fish screening and relocation requirements
- Army Corps Section 404 permit and corresponding environmental documents that were obtained in 7 months from application.
- steel sheetpile shoring was prescribed for this sewer installation across the creek and through the adjacent wetland to control groundwater inflow within the excavation and to maintain groundwater level in the wetland
- all work within the creek, including installation of auger-cast piles and pile caps, sewer pipe, restoration, and removal of sheetpiles and creek bypass to be completed within the approved fish “window” from July 1 to August 31 (allowed construction period from environmental permits).

Retaining wall along Interstate 405. Further upstream, the selected alignment heads north along 114th Ave SE, which is adjacent to a 30 feet high retaining wall holding up I-405, the only north-south highway that provides access into Bellevue. Due to the existing utilities (including a 23 conduit communications ductbank) along the western half of 114th Ave SE, the new sewer alignment runs along the east side of the roadway, up to 24 feet deep and within 10 feet of the retaining wall. Construction of the new sewer will have to mitigate for any potential movement of the retaining wall, yet also have enough clearance from the existing utilities.

In response to this risk, typical trench box shoring was not allowed adjacent to the retaining wall and special shoring (e.g., slide rail system) to minimize lateral ground movements was prescribed where the bottom of excavation was within a 1:1 slope from the bottom of the footings of the retaining walls. The contractor is also required to submit a shoring work plan and calculations, subject to review by the City of Bellevue and the Washington State Dept. of Transportation (WSDOT). As an additional precautionary measure, the contractor is also required to monitor for potential movement of the shoring system and retaining wall during installation of the sewer, with “action” levels in the event displacement thresholds are exceeded.

Getting Across Interstate 405. Another project challenge was getting the new sewer across the 10 lanes of I-405 onto the east side of the freeway. Any crossing of the freeway would require a WSDOT permit and a trenchless solution, as an open-cut installation across I-405 would not be granted. Due to the required invert elevations for the new sewer, any trenchless crossing would have minimal cover (no more than 15 feet) and through alluvial soils.

Initially, two crossing locations were identified as potential locations for crossing I-405. The first location was at the existing 10” sewer crossing. The second location was at the northern end of the project alignment, just north of the Main Street bridge that crosses over I-405 (Figure 1). This location provided more depth of cover than the existing location (15 feet vs. 10 ft). However, the ground conditions were not ideal near the crossing invert as they consisted of loose, alluvial soils that presented a risk of overmining during the tunneling and hence settlement of I-405 above. After the initial evaluation of crossing options, it became apparent that a trenchless crossing of I-405, regardless of location, would involve considerable risk due to the minimal depths of cover and poor ground conditions for trenchless methods. Therefore, additional as-built research was performed to determine whether a potential casing pipe was installed when the original 10” sewer was laid across the freeway. An initial search of both City of Bellevue and WSDOT archives were not successful in finding any record drawings. As a “last ditch” effort, the project team had inquired whether there was anyone within the City that had a long history in the Utilities department that might recall any details of when and how this sewer was installed. Fortunately, this “last ditch” effort worked and a person was found that had access to the original drawings which indicated that, indeed, an 18-inch steel casing pipe was to have been installed for the existing 10-inch sewer. As a precautionary measure after this “discovery”, the project team excavated down to the existing casing pipe and confirmed that a 24-inch casing pipe was actually installed vs. the 18-inch casing that was shown on the drawings (Figure 4). Due to the presence of the existing 24-inch casing pipe, the team decided that a new trenchless crossing of I-405 was not required, and that a structural liner could be installed within the existing casing using a CIPP (cured-in-place pipe) method. Key elements for this CIPP work included:

- sewer bypass system that required WSDOT permit approval for installation of temporary sewer bypass pipe across Main Street bridge over I-405
- verification of size and condition assessment of existing steel casing pipe for feasibility of CIPP
- design of permanent CIPP structural liner (12 mm thickness) to handle ground and hydrostatic loads without casing pipe.

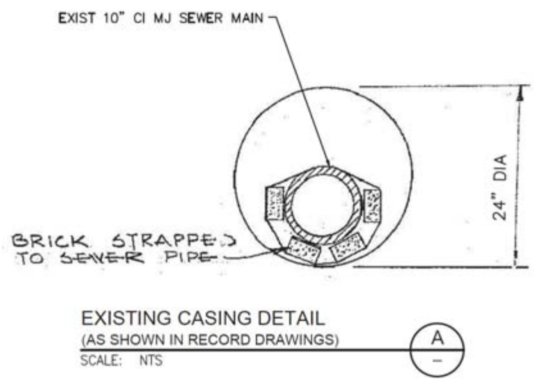

Figure 4. Existing casing detail

Construction through a Lexus car dealership. After crossing I-405 onto the east side, the new sewer runs along two private properties along an existing sewer easement dating back to 1962. Since the time of the original sewer installation, these properties have been further developed and most recently within the past decade, a new Lexus car dealership had been opened. Construction of the new sewer will require close coordination with Lexus operations, and ensuring their vehicle deliveries, car wash, and service customers are not impacted (Figure 5).

Figure 5. New sewer alignment through Lexus parking lot

The existing sewer runs along the western side of the Lexus dealership property, adjacent to the WSDOT right-of-way line. In order to remain within the existing sewer easement, the new sewer had to be located either on top of the existing sewer or further to the east. However, locating the new sewer to the east would have resulted in major impacts to the existing car dealership "service" drive and brought the sewer too close to an existing storm drainage vault. Trenchless methods were also evaluated that would minimize impacts to the car dealership; however, due to dense glacial soils, cost-effective methods such as pilot-tube microtunneling were not feasible. Other methods such as microtunneling, pipejacking, and auger boring were ruled out due to their prohibitive costs as compared to open-trench installation. As a result, an open-cut installation was chosen in the same location of the existing sewer to provide a cost-effective solution that minimized impacts to the car dealership. Key elements for this segment included:

- pumped sewer bypass system for anticipated 3-months construction duration
- requirement to maintain car dealership "service drive" at all times for car dealership customers and service deliveries
- temporary removal and re-installation of car dealership light poles, and provision for temporary lighting during construction
- restoration of all landscaping per previously approved City permits for car dealership construction
- construction within car dealership property restricted from July 1 through Labor Day during major Lexus summer sales event.

The final hurdle. The last major challenge was designing a new 12-inch diameter sewer across Main Street, a major east-west arterial feeding into downtown Bellevue. The new sewer is approximately 30 feet deep and 174 feet in length across Main Street, and any impacts to Main Street had to be minimized (Figure 6). Furthermore, a high groundwater table in this area also required a solution that eliminates the need to dewater across Main Street.

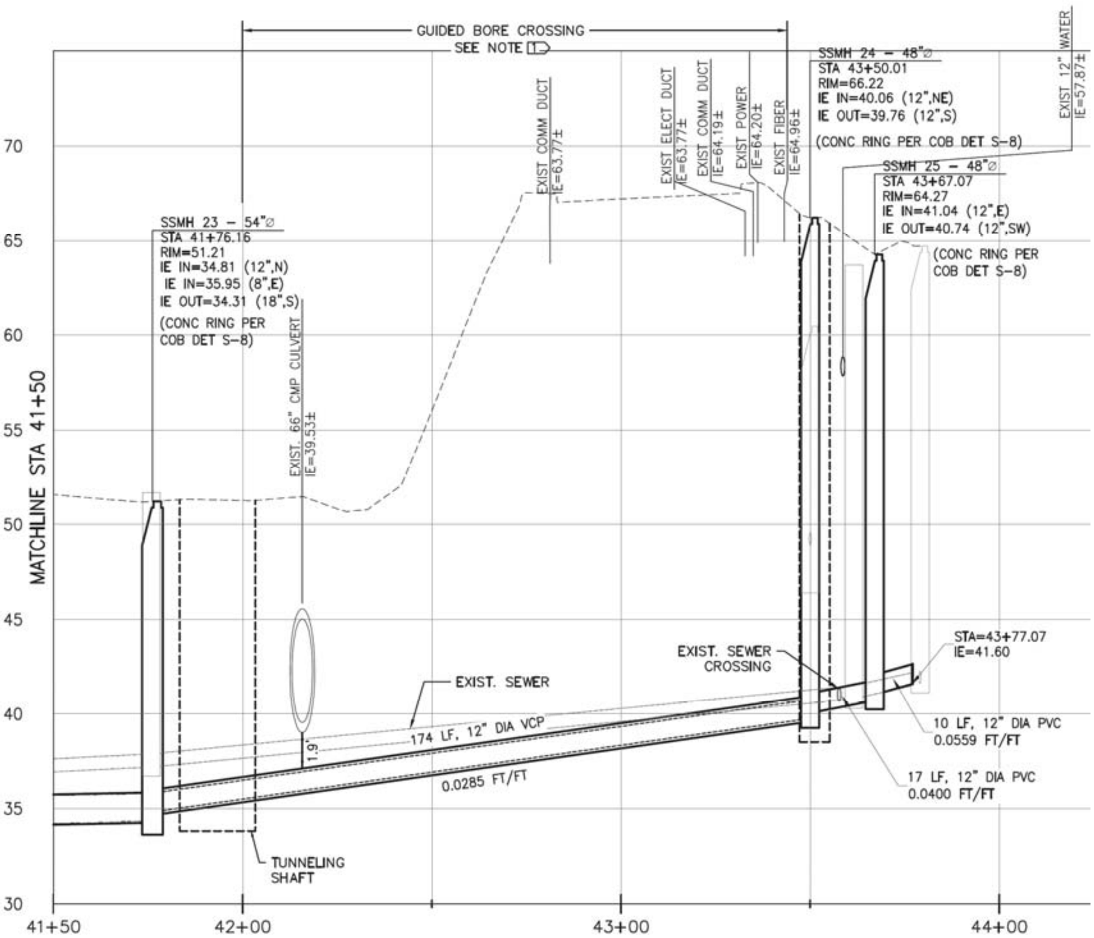

Figure 6. Profile of guided bore crossing of Main St.

Once again, trenchless methods were evaluated for this deep crossing. Due to the high groundwater table, any methods that required dewatering across Main Street were eliminated from consideration, including auger-boring and open-shield pipejacking. Microtunneling was ruled out due to the prohibitive cost for this small diameter, short crossing, and pilot-tube microtunneling was not feasible due to the dense glacial soils. Pipe ramming was a potential method, but was not favorable due to the high impact pneumatic hammer required and associated noise impacts to the adjacent hotel and car dealership. Ultimately, guided-boring technology was selected due to its ability to handle groundwater, cost-effectiveness, and ability to control line and grade. One of the concerns of this method was the risk of getting "stuck" under Main Street due to obstructions such as cobbles and boulders in the glacial drift. Therefore, a contingent bid item was placed in the bid documents for removal of obstructions in the event the guided bore crossing encounters an obstruction (e.g., a bouder) that stops the forward progress of the bore.

Conclusion

Although the Wilburton Sewer Improvements project is not an uncommonly long sewer alignment at 4,300 lineal feet, the project has numerous and very distinct challenges along every segment of its corridor. These challenges provided an opportunity for the project team to evaluate various alternatives for addressing each particular situation, and to ultimately select a wide variety of solutions to complete the design including pile supported pipe, an open-cut crossing of a creek, prescriptive shoring methods for the contractor, and cured-in-place pipe and guided boring trenchless methods. With construction starting in the summer of 2015, the project team eagerly waits for its design solutions to be tested against the challenging project conditions.

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Potts Ditch: Rerouting the Impossible

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Abstract

Located in the City of Greenfield, Indiana, Potts Ditch is a partially encapsulated stream that was constructed over a century ago. The encapsulated infrastructure is constructed mostly of brick arches and is reaching the end of its useful life. Portions of the encapsulated infrastructure are located underneath and adjacent to existing buildings, thereby putting these structures at risk. Furthermore, the ditch is undersized and contributes to flooding in the downtown area. The project, currently under construction, includes installation of approximately 2,000 linear feet (LFT) of new 14-foot by 6-foot precast concrete box sections to reroute Potts Ditch within the City right-of-way. The stream was modeled in HEC-RAS. Design included a detailed sequence of construction to install the proposed facilities while maintaining existing operations, minimizing the need for bypass pumping, and minimizing the impact to the affected neighborhoods. The affected street corridors are being completely rebuilt and utilities are being relocated to accommodate the construction. Due to vertical conflicts between the existing gravity sewers and the precast concrete box, the City decided to proceed with a 24-inch sanitary sewer interceptor project through the project area to maintain gravity sewer service. Subsurface Utility Engineering Quality Level A was completed in strategic locations. At project completion, the City will have improved storm drainage, filled in a brick arch that is at the end of its useful life, extended a sanitary sewer interceptor, and reconstructed street corridors within the project area.

INTRODUCTION

Located in the City of Greenfield, Indiana (City), Potts Ditch is a partially encapsulated stormwater conveyance stream that was constructed over a century ago and is reaching the end of its useful life. Segments of Potts Ditch are located underneath and adjacent to existing buildings and roadways – including two state roads – putting these structures at risk. Furthermore, the stream is undersized and contributes to flooding in the downtown area of the City.

The City is undertaking a project to relocate the encapsulated Potts Ditch using precast concrete box structures along city streets to improve stormwater conveyance and access. Due to the size and tight urban location of the project area, many existing pipelines are affected. The proposed precast concrete box interferes with the existing gravity sanitary sewer and laterals. To resolve this conflict, the City is extending a new sanitary sewer interceptor through the project area.

The City retained American Structurepoint, Inc. (American Structurepoint) for the design of the improvements. This paper presents a case study of the planning and design incorporated into the project to achieve a workable and constructible solution. Some of the major challenges – which made the project seem daunting or even impossible – are also described.

HISTORY AND BACKGROUND

Potts Ditch was originally constructed approximately a century ago of brick and stone arches. A photograph of the original brick arch crossing underneath North Street is included in Figure 1.

Figure 1. Photo of Original Potts Ditch at North Street

In 1990, the City rehabilitated the entirety of the encapsulated Potts Ditch using reinforced shotcrete to repair walls and foundations, many of which were deteriorated and were missing bricks (see Figures 2 and 3). In 2002, Hancock County replaced the portion of the structure from Main Street to the South Street with 14-foot by 6-foot cast-in-place concrete box to reroute Potts Ditch around a new County Community Corrections building.

Figure 2. Photo of Missing Bricks in Wall at North Street, Pre-Rehabilitation

Figure 3. Photo Showing Shotcrete near North Street, Year 2013

The dimensions of the existing encapsulation vary non-uniformly in the project area, resulting in 16 unique cross sectional areas ranging from approximately 38 square feet to 84 square feet. The capacity of the existing infrastructure varies between an estimated 50 and 610 cubic feet per second (cfs) when flowing full (Clark Dietz, 2012). As a result, the encapsulated stream creates a hydraulic bottleneck.

The existing Potts Ditch alignment and project limits are shown in Figure 4. The stream flows south through an open channel section before entering the encapsulated section. Existing Potts Ditch flows underground through the downtown area of the City, crossing underneath several buildings, city streets, Indiana State Road 9, and US Route 40. The stream reemerges as open channel flow in two areas adjacent to North Street. The project area ends at South Street, where the stream returns to open channel flow.

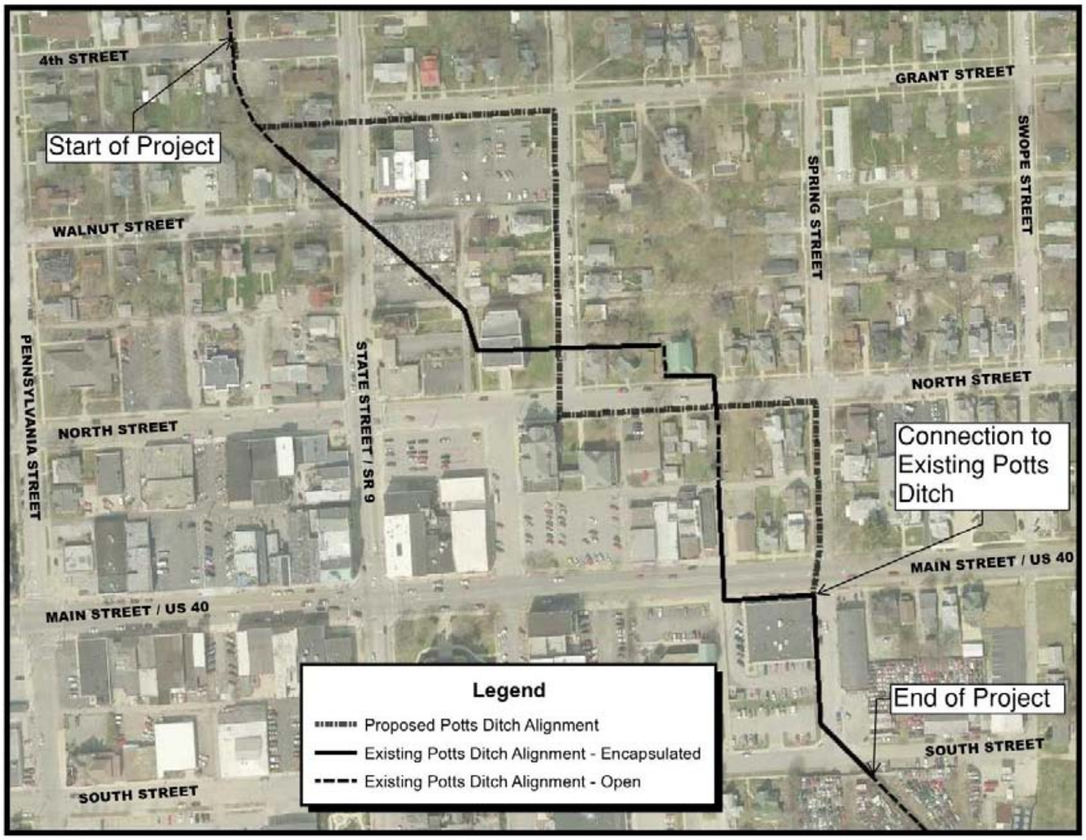

Figure 4. Project Location Map

STUDY PHASE

In 2012, the Potts Ditch Storm Drain Improvements Study (Study) was undertaken by the City and completed by Clark Dietz, Inc. They developed a hydrologic model using the United States Army Corps of Engineers (USACE), Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS). The hydrologic model was used to estimate current and future peak flows in the Potts Ditch Watershed, and the results are summarized in Table 1.

The Study recommended replacement of the existing encapsulated Potts Ditch with either a 14-foot by 6-foot concrete box or dual 8-foot by 6-foot concrete boxes in parallel to match the capacity of the 14-foot by 6-foot section at the downstream end of the encapsulation. The Study stated either of these options has capacity to convey the current 10-year storm, assuming existing detention is maintained. The new Potts Ditch capacity in relation to the modeled peak flows is discussed further in the Stream Modeling section.

Table 1. HEC-HMS Model Results Summary (Clark Dietz, 2012**)**

Scenario	Peak Flow (cfs)
Current Conditions, Including Existing Detention, 10-year Storm	556
Current Conditions, Including Existing Detention, 100-year Storm	1,253
Current Conditions with Future Upstream Bypass, 100-year Storm	575
Future Conditions, Post-Development, 100-year Storm	637

STORMWATER CONVEYANCE DESIGN

Stream Modeling. To demonstrate the changes to Potts Ditch would not increase 100-year water surface elevations by more than 0.14 feet per Indiana Department of Natural Resources (IDNR) standards, American Structurepoint modeled this stream reach using the Hydrologic Engineering Center River Analysis System (HEC-RAS). The model for the 1981 City of Greenfield Flood Insurance Study (FIS) – truncated to the project area for this study – was used as the basis for developing the HEC-RAS Corrected Effective and Post-Project models.

Because of hydraulic restrictions of the existing encapsulation, during the 100-year storm a portion of the stormwater enters the encapsulation, and the remaining stormwater flows overland. The encapsulation was not included in the original FIS model, so the Effective model did not reflect the culvert/overland split portion of Potts Ditch. Instead water surface elevations were set at cross sections downstream and upstream of the encapsulation, and the results were compared with the FIS to verify the HEC-RAS Corrected Effective model matched the original.

In the Post-Project model, both the new 14-foot by 6-foot encapsulation and overland flow portions of Potts Ditch were incorporated. In order to model the flow split, the Federal Highway Administration HY-8 culvert analysis program was used to calculate the capacity of the encapsulation at various headwater elevations. A headwater elevation of 880.6 feet results in a capacity of 747 cfs, and represents the elevation at which water begins to flow overland at the upstream headwall based on existing topography. This capacity was used for the encapsulated flow reach in HEC-RAS, and the remaining 100-year peak flow (1,103 cfs ignoring existing detention) was routed through the overland reach for the Post-Project model. A capacity of 747 cfs for the new encapsulation is a conservative estimate during the 100-year event, because it ignores additional capacity caused by the increased headwater.

The analysis showed the 100-year water surface elevation from the proposed Potts Ditch improvements reduced flood elevations immediately upstream of the headwall by approximately 9-inches, and therefore was acceptable to IDNR. In addition, the capacity of 747 cfs confirmed the Study conclusion that the existing 10-year storm can be conveyed by the new encapsulated Potts Ditch, assuming existing detention is maintained (see Table 1 [Clark Dietz, 2012]). Although there is insufficient capacity to convey the 100-year storm based on existing conditions, a future bypass of a portion of the upstream flow would reduce the peak flow to levels that can be conveyed. Alternatively, because future development is restricted to lower peak flow than existing undeveloped ground, the post-development 100-year storm could be conveyed by the new Potts Ditch without any bypass. For these reasons, the modeling confirmed that 14-foot by 6-foot precast concrete box is acceptable.

New Potts Ditch Encapsulation. In addition to the main goals to improve stormwater conveyance and replace aging infrastructure, the City also wanted to relocate Potts Ditch into city street rights-of-way to allow for better access. The proposed route for the 14-foot by 6-foot precast concrete box is shown in Figure 4, and it totals approximately 2,000 linear feet. The alignment starts just south of Fourth Street and follows Grant Street, East Street, North Street and Spring Street. The culvert underneath Fourth Street is also being replaced. The route was selected to limit the impacts to the thoroughfares of State Road 9 and US Route 40. By crossing these roads rather than running parallel, the closures are limited to between two and four weeks at a time rather than several months. The route stays as close to the existing encapsulation as possible, thereby limiting the impact of new storm sewer trunk lines required to convey flow from existing inlets.

Precast concrete was selected rather than cast-in-place to reduce the impact of excavation by allowing a faster turnaround between excavation and backfill. The proposed Potts Ditch incorporates a 14-foot by 6-foot precast concrete box rather than dual 8-foot by 6-foot precast concrete boxes (the other option recommended in the Study) primarily for the following reasons:

- There is limited space in the city street corridors. The dual boxes would require more space (18.7 feet minimum width for dual boxes butted against each other versus 16 feet) and require additional excavation.
- The dual boxes would require additional maintenance access points.
- The dual boxes would require twice as many joints.

City right-of-way is as narrow as 50 feet in the project area. Fitting the proposed infrastructure in the right-of-way was one of the main challenges since there are sanitary sewers, water mains, gas mains, telecommunication duct banks and overhead power lines in the area. As an example, Figure 5 shows the underground infrastructure within the Spring Street right-of-way. The proposed precast concrete box structure was kept close to curb lines to allow for space for utilities and construction.

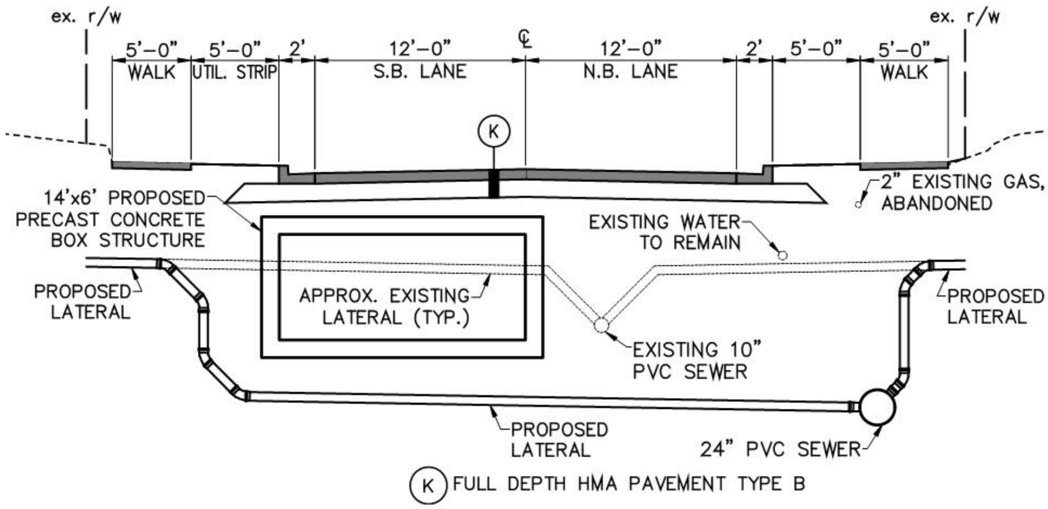

Figure 5. Spring Street Typical Section

Several cast-in-place concrete structures were required as connection structures for the precast concrete box structure. The start of the precast box at Main Street/US 40 required tying into the existing 14-foot by 6-foot structure. The cast-in-place structure provided at this location allows for Potts Ditch flow to be maintained through the existing infrastructure while also allowing flow from storm inlets to be connected to the new precast concrete box as construction progresses.

Typically, horizontal bends made by precast concrete utilize a series of wedge-shaped pieces, thereby increasing the bend radius and footprint. In order to save space, the 90 degree bends in the Potts Ditch alignment were accomplished by using cast-in-place structures. The City noted that Potts Ditch attracts illegal disposal of large rubbish, creating hydraulic restrictions and increasing flooding. In order to prevent the debris from entering the new precast concrete box, a manual bar screen is being provided at the new upstream cast-in-place headwall. Potts Ditch has also attracted trespassers inside of the existing encapsulated stream. To prevent people from entering on the downstream end, a similar bar screen is also provided at the existing downstream headwall at South Street.

Conflict Structures. On both North Street and East Street, the new precast concrete box sections cross the existing encapsulated Potts Ditch. The precast sections could not be installed through the existing ditch without bypassing the flow. Options were considered to reduce the need for bypassing. One option was leaving the brick arch in place at the conflict, continuing to set precast sections on the other side, and bridging the gap at the end of the project. During construction, it was determined to tie in the existing brick arch with the new precast concrete box immediately as construction passed the conflict point. The plan includes providing a metal arch tying into a 3-sided precast section, with grout around the arch to make it water tight. After the final upstream connection is made the City's contractor, Renascent, Inc. (Renascent), will remove the metal arch and cast the fourth wall of the box section in place as a final step. This plan allows more work to be completed concurrently with the main construction in the area, reducing the amount of time and impact during the final step of construction.

Storm Sewers. Because the City does not have significant problems associated with surface drainage (the flooding problems are mostly due to the undersized Potts Ditch infrastructure), the number of existing storm inlets was maintained. Because the new Potts Ditch alignment does not follow the existing alignment exactly, some storm inlets needed to be rerouted with new trunk lines. The inlets at the intersection of Walnut Street and State Street/State Road 9 (see map in Figure 4) were rerouted through the alley extending east from Walnut Street, picking up additional inlets in the alleys in the area before tying into the new Potts Ditch alignment at East Street. Using the alleys reduced the impact to State Road 9.

Potts Ditch Abandonment. Due to the risks associated with an existing open structure under buildings and roadways, the City determined they would fill in existing Potts Ditch rather than abandoning it in place. The fill will be completed by using either flowable fill or cellular grout. The open channel sections will be filled with soil and graded as a swale to drain.

Permitting. Because the open channel portions of Potts Ditch are considered Waters of the US and the project would impact these areas, a USACE Section 404 Regional General Permit (RGP) and an Indiana Department of Environmental Management Section 401 Individual Water Quality Certification were required. The original intent of the project was to fill the existing open channel portions near North Street and encapsulate the existing open channel section between the Fourth Street culvert and the existing start of encapsulation (see Figure 4). However, the proposed encapsulation between the Fourth Street culvert and the existing start of encapsulation would have pushed the project over the USACE RGP limit of 300 LFT of channel loss before compensatory mitigation is required. To minimize impacts, avoid mitigation, reduce project costs, and reduce overall project timeframe this open channel section remained open.

In addition, as part of the USACE permitting process an Archaeological Records Search and Literature Review and an Identification of Effects Report was completed. The project occurs within the national Register of Historic Places - listed Greenfield Residential Historic District. In coordination with the State Historic Preservation Officer (SHPO) it was determined that the impacts to the contributing resources within the project area would not diminish the historic district’s ability to convey its significance in the areas of architecture and community development.

GEOTECHNICAL INVESTIGATION

The geotechnical investigation was completed by CTL Engineering, Inc. The results of the investigation indicated that the “on-site excavated soils, except for topsoil, organically contaminated soils and construction type debris, are considered suitable for use as backfill material provided proper moisture content is maintained during placement (CTL, 2014).” The option of using either on-site soils compacted to 98 percent of maximum dry density or Indiana Department of Transportation (INDOT) Standard Structure Backfill was given to bidders. Renascent estimates that the use of on-site soils rather than borrow material for backfill saved the City approximately $1,000,000.

CITY-OWNED UNDERGROUND UTILITIES DESIGN

Only design of City-owned underground utilities was included in the project. However, coordination was also required for relocations of gas, telecommunications, and City-owned overhead electric utilities. It was private utilities' responsibility to relocate if they were in City right-of-way and in conflict with proposed construction.

Subsurface Utility Engineering (SUE). Due to the extensive excavation required to install the new precast concrete box sections, impact to existing underground utilities could be significant. SUE investigations provide data during design to identify required relocations and limit surprises during construction. American Structurepoint retained Cardno, Inc. to complete SUE Quality Level B investigation in the entire project area as a first step in verifying the utility locations previously identified by the Indiana 811 Utility Location System (Indiana 811). During the investigation, Cardno, Inc. identified several additional utilities and corrected utility locations previously identified by Indiana 811. In addition, test holes (SUE Quality Level A) were completed in locations where:

- Unknown utilities were found during Quality Level B investigations and needed to be verified,
- Discrepancies existed between the mapping and the location identified during SUE, and
- Existing depth was critical to the design (e.g. at a proposed crossing).

In one instance, the water main marked by Indiana 811 and the location determined by SUE Quality Level B were on opposite sides of East Street. The Quality Level A test holes verified that the Quality Level B location was accurate and that the previously identified line was not an active water main.

Sanitary Sewer. Because the proposed Potts Ditch is deeper than some of the existing gravity sewers in the area, there are conflicts with the existing laterals (see Figure 5). A 15-inch sanitary sewer interceptor runs directly underneath the flow line of existing Potts Ditch for the majority of the project area, resulting in limited access to the interceptor and the increased likelihood for inflow and infiltration. The existing interceptor increases to 24-inch diameter at South Street. To serve the project area by gravity and allow easy access to the interceptor, the City elected to replace the existing interceptor with approximately 2,600 LFT of new 24-inch PVC interceptor.

The new interceptor alignment parallels the precast concrete box structure. Due to space constraints within the City rights-of-way, laterals were allowed to connect directly to the new interceptor without first connecting to smaller gravity sewers. The laterals have minimal vertical separation from the new precast concrete box at the downstream end of the project, but the clearance increases upstream.

On East Street, the proposed sewer interceptor crosses the existing interceptor, which is located underneath the brick arch of Potts Ditch (see Figure 6). This posed a problem in maintaining flow in the existing interceptor. Typically this is accomplished by bypass pumping around the conflict point. However, the remainder of the interceptor upstream within the project area is also underneath Potts Ditch. Therefore, bypass pumping would require exposed sanitary sewer piping to pass vertically through Potts Ditch and be in contact with periodic stormwater flows for the remaining months of construction. Therefore, to limit environmental impacts, the connection shown in Figure 6 was designed to connect the existing interceptor to the new interceptor until the final upstream connection is made.

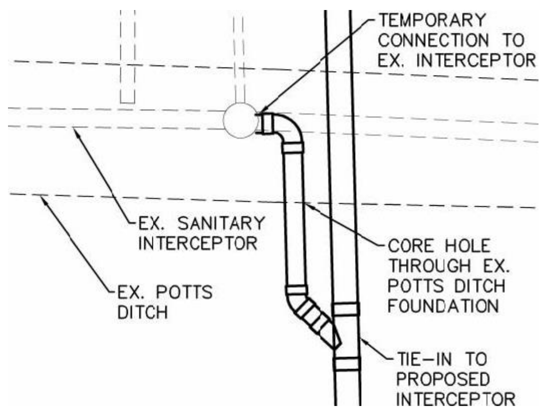

Figure 6: Temporary Interceptor Connection Plan in East Street

Water Mains. There are several areas where the new Potts Ditch alignment crosses existing water mains, resulting in vertical conflicts. There is insufficient cover on top of the precast concrete box, so the new water mains and services pass underneath the box. The water mains are being installed in a casing underneath the Potts Ditch, so that they can be removed if necessary in the future. The water mains on North, East, and Grant Streets are being replaced in their entirety to improve distribution looping.

ROADWAY DESIGN

Because of the amount of disruption to the city streets in the project area, the streets are being completely rebuilt including curb and gutter and sidewalk. The roads are mostly being replaced in kind with only minor changes to profile and cross sections. The amount of heavy construction and road work in the project prompted American Structurepoint to use the INDOT Standard Specifications, with special provisions added for water main and sewer work. The INDOT Standard Specifications are well defined for heavy construction and are familiar to contractors completing this type of work.

CONSTRUCTABILITY AND CONSTRUCTION

During design, the design team and construction inspection personnel of American Structurepoint reviewed the project to verify constructability. It was determined that the construction would impact the building on the west side of the intersection of Grant Street and State Street/SR 9. The City decided to purchase and remove the building to facilitate construction.

A detailed suggested construction phasing was provided as part of the bidding documents. It outlines the intent of the design to demonstrate constructability and establish acceptable levels of service, while leaving means and methods up to the contractor. The suggested phasing explains the engineers' intent for reducing Potts Ditch bypass pumping. Specific construction constraints – such as limits to the amount of unrestored construction – were differentiated from suggestions in the document. The goal was to reduce bidder uncertainty.

The City awarded the construction contract to Renascent, Inc. (Renascent) in June 2014. American Structurepoint is performing on-site construction inspection. The project is currently under construction, and has progressed from the downstream end onto North Street as of January 23, 2015. The scheduled completion is the end of 2015. Figure 7 is a photo of the precast concrete box installation.

Figure 7. Precast Concrete Box Installation on North Street

As with any public works project of this magnitude, public participation is a key to success. The project is disruptive by nature, so the goal is to keep the disruption to an acceptable level by keeping the public informed. At the start of construction an open-house was held where members of the public could ask questions and voice concern to members of the City, American Structurepoint, and Renascent. Informal weekly public meetings are convened each Tuesday morning to inform the public of progress. In addition, the Mayor goes door-to-door to meet with residents and business owners before the construction progresses into their area to address their concerns and specific needs.

SUMMARY

The Potts Ditch relocation is one of the largest public works projects in City history. The project required integration of many disciplines including hydrology, stormwater conveyance, sanitary sewer, water main, subsurface utility engineering, roadway, geotechnical, and structural. The project includes approximately 2,000 LFT of 14-feet wide by 6-feet deep precast concrete box structure, 2,600 LFT of 24-inch polyvinyl chloride (PVC) gravity sanitary sewer interceptor, and 2,000 LFT of various sizes of water main. Affected street corridors are being rebuilt and the existing encapsulation is being filled after the new encapsulation is completed. Public participation and acceptance were essential during design and are on-going during construction. The project is currently under construction, and it is scheduled for completion by the end of 2015. At the end of the project, the City will have improved stormwater conveyance, improved a sanitary sewer interceptor, and retired a century-old brick storm drain.

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