PIPELINES 2007

Advances and Experiences with Trenchless Pipeline Projects

PROCEEDINGS OF THE ASCE INTERNATIONAL CONFERENCE ON PIPELINE ENGINEERING AND CONSTRUCTION

July 8–11, 2007
Boston, Massachusetts

SPONSORED BY
The Pipeline Division of the American Society of Civil Engineers

EDITED BY
Lynn Osborn, P.E.
Mohammad Najafi, P.E.

ASCE American Society of Civil Engineers
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Foreword

The Pipeline Division of the American Society of Civil Engineers (ASCE) is pleased to present the *Proceedings* of the **Pipelines 2007** International Conference, entitled **"Advances and Experiences with Trenchless Pipeline Projects**." This conference was held July 8-11, 2007, at the Westin Boston Waterfront Hotel in Boston, Massachusetts.

Pipelines 2007 was a well-attend and memorable event in the history of ASCE Pipeline conferences in which approximately 120 papers were presented. The conference technical program focused on application of existing and emerging technologies for asset management, trenchless technologies, condition assessment, pipeline renewal, pipeline protection, and risk assessment. In addition, five workshops on pipeline and manhole coating and lining systems, emerging concepts on pipeline renewal design, design and construction of pipe ramming projects, plastic pipe design concepts for water systems and standardizing asset management and repair for municipal infrastructure were presented.

The success of this conference was the result of volunteer work and efforts of many pipeline professionals. Most of all, the **Moderators and Reviewers** played a major role by organizing sessions, and participating in the submission and review process of the papers. The **Conference Steering Committee** worked for two years to plan and organize the conference.

All papers published in these *Proceedings* were reviewed by at least two pipeline professionals, including the **Session Moderators and the Conference Steering Committee** members. All papers are eligible for discussion in the ***ASCE Journal of Transportation Engineering*** and are also eligible for **ASCE Awards**. Appreciation is extended to all who worked so hard to bring this exceptional conference together. The high quality of papers presented is due to obvious efforts of the **Authors, Moderators, and Paper Reviewers.**

The dedication and hard work of ASCE staff, especially **Elaine Watson**, ASCE Conference Manager, **John Segna**, Director, Technical Activities Department and **Donna Dickert**, Manager, Proceedings Production, are greatly appreciated.

Special thanks go to all sponsors of the conference including the **Center for Underground Infrastructure Research and Education (CUIRE) at the Department of Civil and Environmental Engineering at The University of Texas at Arlington** for providing the resources and support for this conference. In particular, we appreciate the efforts of **Behnam Hashemi**, CUIRE Research Assistant, who spent many long hours coordinating the review process.

On behalf of the Conference Steering Committee,

Lynn Osborn and Mohammad Najafi	John Struzziery and Tim Stinson
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Atlanta Track – Paper One

Atlanta's Consent Decrees Drive a Substantial Commitment to Trenchless Sewer Rehabilitation

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Abstract

In 1998 and 1999, the United States Environmental Protection Agency (EPA) and Georgia Environmental Protection Division (EPD) completed settlement negotiations associated with two complaints against the City of Atlanta for various violations of the United States Clean Water Act and Georgia Water Quality Act. This resulted in the recording of the Consent Decree (CD) related to Combined Sewer Overflows (CSOs) and the First Amended Consent Decree (FACD) associated with Sanitary Sewer Overflows (SSOs). They mandated improvement of the City wastewater collection and transmission systems including wet-weather control for combined sewers, and rehabilitation and relief of capacity-limited sanitary sewers. The City's seven existing CSO control facilities experience a combined 300 discharge events per year. Additionally, over 550 SSOs (unpermitted discharges and spills to dry land) are experienced annually.

The City wastewater collection and transmission systems include approximately 1600 miles of sewer mains and approximately 37,500 manholes. Fifteen percent of the system is comprised of combined sewers while 85 percent of the system consists of separate sanitary sewers. Six other governmental entities contract for wastewater treatment by the City including the Cities of Hapeville, College Park and East Point, and DeKalb, Clayton and Fulton Counties. Fifty-five percent of the sewage flows are generated by Atlanta and 45 percent by the wholesale agencies. The total benefiting population is 1.2 million.

The 14-year phased approach under the FACD includes evaluation and rehabilitation of the sanitary system, and construction of capacity relief sewers that is estimated to cost approximately $2.2 billion. Additionally, the CD requires a $1.0 billion capital investment including disinfection improvements, new tunnel conveyance/storage and treatment facilities, and sewer separation in three combined sewer basins. Additionally, the FACD requires the preparation and implementation of management, operation and maintenance ("MOM") programs for the sanitary sewer system as a measure to help control SSOs. Full compliance with the CD and FACD is required by November 7, 2007 and July 1, 2014, respectively.

Over 200 miles of small diameter sewers have been rehabilitated to date for the most part utilizing cured in place pipe. The City anticipates escalating its rehabilitation program to an average annual expenditure of $100 million over 8 years.

This paper will overview the City's challenges with negotiating complex consent decrees with EPA/EPD and complying, to date, with some of the scheduled 900 interim and final consent decree compliance milestones. The paper will also describe the formation of the improvement program, locally known as 'Clean Water Atlanta', that was formed to meet the requirements of the FACD and will also introduce the three other papers in this track relating to SSES and Rehab Selection Process (Paper Two), The GIS Hub (Paper Three), and Hydraulic Modeling (Paper Four)

Introduction

In 1998 and 1999, the United States Environmental Protection Agency (EPA) and Georgia Environmental Protection Division (EPD) issued consent decrees addressing the City of Atlanta's combined sewer overflow and sanitary sewer overflow compliance issues, respectively. This allowed for the dismissal of two complaints against the City for various violations of the United States Clean Water Act and Georgia Water Quality Act.

The Consent Decree (CD) deals with combined sewer overflows (CSOs) while the First Amended Consent Decree (FACD) tackles sanitary sewer overflows (SSOs). They mandate improvement of the City wastewater collection and transmission systems including wet-weather control for combined sewers, and rehabilitation and relief of structurally compromised and capacity-limited sanitary sewers. The City's six existing CSO control facilities currently experience a combined 300 discharge events per year. Additionally, approximately 600 SSOs (unpermitted discharges to waters of the state and spills to dry land) have been experienced annually during the period 2002 through 2005, down from 1,000 spills in 2000.

This paper will: 1) overview the City's challenges in negotiating and complying with its consent decrees; 2) describe the goals and objectives of the "Clean Water Atlanta" initiative; and, 3) introduce the three other papers in this track relating to SSES and

Rehab Selection Process (Paper Two), the GIS Enterprise "Hub" (Paper Three), and Hydraulic Modeling (Paper Four).

Background

The City wastewater collection and transmission systems include approximately 1,900 miles of sewer mains and laterals in rights-of-way and easements, and approximately 40,000 manholes. Fifteen percent of the system is comprised of combined sewers while 85 percent of the system consists of separate sanitary sewers. Six other governmental entities contract for wastewater treatment by the City including the Cities of Hapeville, College Park and East Point, and DeKalb, Clayton and Fulton counties. Fifty-five percent of the sewage flows are generated by the City of Atlanta and 45 percent by the wholesale agencies. The total benefiting population is 1.6 million.

The 14-year phased approach under the FACD includes evaluation and rehabilitation of the sanitary system, and construction of capacity relief sewers that is estimated to cost approximately $2.2 billion. Additionally, the CD requires a $1.0 billion capital investment including disinfection improvements, new tunnel conveyance/storage and treatment facilities, and sewer separation in three combined sewer basins. Furthermore, the FACD requires the preparation and implementation of management, operation and maintenance ("MOM") programs for the sanitary sewer system as a measure to help control SSOs. Full compliance with the CD and FACD is required by November 7, 2007 and July 1, 2014, respectively.

Consent Decrees – Challenges in Negotiating and Complying

Negotiations

During negotiations of both consent decrees two common areas of intense focus were experienced during final stages of negotiations including: 1) development of a mutually agreeable schedule for achieving interim and final compliance with all milestones; and 2) formulation of a "reasonable" schedule of civil and stipulated penalties.

<u>Schedule.</u> Ultimately, a 9-year compliance schedule was included in the CD. However, a provision allows for preparation of a financial capability analysis that could possibly allow an extension of the full compliance deadline based on affordability to the rate payers. The City has been pursuing such relief unsuccessfully for over three years as a result of concluding a high burden of compliance on its customers where over 20 percent of the households are at or below the poverty level. With regard to the FACD, originally a 16-year compliance deadline was pursued by the City. Tough negotiations in 1999 resulted in a 14-1/2 year compliance period.

<u>Penalties.</u> A total of $3.2 million in civil penalties and $4.3 million in stipulated penalties have been assessed against the City to date. Payment of these penalties was

remitted to the federal and state treasuries with no direct benefit necessarily to the environment of the Atlanta metropolitan area. However, under terms of the CD, in

lieu of additional cash penalties to be directly remitted to the governments, the City will ultimately expend $25 million in the acquisition of protective greenway buffer adjacent to select Atlanta waterways as part of a negotiated Supplemental Environmental Project (SEP). Additionally, the City committed over $5 million to a second SEP, namely, creek cleanups where auto tires and other debris were illegally dumped. Together, these two efforts will go a long way to sustain local improvement and protection of water quality. A similar approach in pursuing a SEP in lieu of the extent of civil and stipulated penalties ultimately to be remitted under the FACD was unsuccessful.

Compliance

Challenges to the City complying with various key consent decree terms have been substantial. Resolution has required creativity and perseverance on the part of the City.

<u>Project Substitution to Ensure Meeting Objectives.</u> The FACD specifically described a wastewater diversion project including a sizeable pump station and long force main requiring acquisition of over 100 parcels in the north of the City. The project would have provided capacity relief in one sewer basin by transferring a substantial amount of wastewater to an adjacent basin. What became apparent after issuance of the FACD was that both basins would still require wet weather capacity relief of major trunks and outfall sewers after constructing the diversion. As a result of a negotiated amendment to the FACD, an 8.5-mile, 16-foot finished diameter conveyance and storage tunnel – the more intelligent project – was substituted accomplishing the required capacity relief with no inter-basin wastewater transfer.

<u>Avoided Penalty for Failure to Meet an Interim Milestone Date</u>. Ten capital sewer projects were included in the negotiated FACD with completion of all required by 2003. Completion of one of the projects, a combination open-cut and micro-tunneled relief sewer, was delayed due to contractor non-performance. While a substantial penalty of over $800,000 was contemplated by EPA/EPD, the City negotiated a new substantial completion date with the relief sewer contractor and a $100,000 penalty with the EPA/EPD with the balance payable and due only if the above mentioned conveyance and storage tunnel project was not completed before January 1, 2006. The relief sewer was substantially completed by the revised completion date and with cooperation of the tunnel contractor, that project was rushed to substantial completion on December 31, 2005, avoiding further monetary penalties and possible land development moratoria.

<u>Compliance Resulting in Possible Conflicts.</u> The FACD was originally negotiated with a staggered schedule for completion of improvements in 6 groupings of collection system sewersheds. A sewershed consists of an average of approximately

33,500 feet of mainline sewer. A term of the FACD required the development by the City of a prioritization plan to schedule the work in these 6 areas, completing the work by 2009, 2011, 2013 or 2014. Utilizing 10 criteria, the plan programmed capacity relief and sewer rehabilitation on the basis of those in worst condition being repaired first. While this was an acceptable approach for purposes of resolving structural defects with those sewers in the greatest state of disrepair scattered around the City to be fixed first, it did not work for capacity relief where upstream hydraulic constraints would not be totally resolved perhaps until downstream hydraulic constraints were addressed as well. The City negotiated successfully with EPA/EPD under existing provisions of the FACD allowing the City to separately analyze and schedule capacity relief in six grouped areas (known as 'Sewer Groups') independently from rehabilitation, as necessary, but with work also to be completed by 2009, 2011, 2013 or 2014.

Clean Water Atlanta - A Five-Point Plan

Since the early 1980s, the City has faced challenges in complying with increasingly stringent federal Clean Water Act standards and state Water Quality Control statutes which ultimately led to the City entering into the two consent decrees.

In response to this latest challenge, on October 16, 2002, Mayor Shirley Franklin announced the Clean Water Atlanta initiative -- the City's comprehensive, long-term plan to ensure clean drinking water for Atlanta, and clean streams and clean wastewater flows for Atlanta and its downstream neighbors -- for the mayor recognized that the City's economic vitality, jobs growth, affordable living for its citizens, and quality of life all depended on clean water.

With the announcement of the initiative, the Mayor unveiled a Five Point Plan for improving the city's wastewater system including:

1. Professional Management of Consent Decree Program

The Department of Watershed Management was created in 2002 to oversee the City's new comprehensive approach to solving water issues. The DWM includes the City's two water-related bureaus – Wastewater Services and Drinking Water – along with Engineering Services and the emerging Stormwater Management Utility. This organizational structure allows DWM to plan, design, construct, operate and maintain the City's entire system of water and wastewater treatment, pumping, collection and distribution, and proposed stormwater management facilities on an integrated basis.

2. Strategy to Reduce Flooding and Pollution Caused by Stormwater

The City's goal is to implement a stormwater utility that will provide a steady and reliable source of revenue for capital projects to reduce stormwater flooding and pollution while maximizing the use of natural pollution-reduction methods such as greenspace and ponds.

3. FACD (SSO Consent Decree) Compliance

Under the FACD, the City is inspecting and repairing, replacing or rehabilitating, as necessary, all 1,900 miles of sewer throughout the City and implementing long-term prevention and maintenance strategies under "Operation Clean Sewer." Goals of Operation Clean Sewer include:

- Clean 25% of the sewer system each year
- Physically inspect 15% of the sewer system each year, and
- Rehab 2% of the sewer system each year
- Inspect permitted grease traps twice each year

The City has far exceeded each of the above mentioned goals which to date is greatly responsible for reducing total spills by 40% since 2000.

4. Water Quality Monitoring to Ensure Effectiveness of Clean Water Atlanta Programs

The City has partnered with the U.S. Geological Survey to implement a comprehensive water quality monitoring plan. Forty stream sites are monitored including twenty sites equipped with automated sampling and telemetry for real time queries for a number of water quality parameters and flow conditions in the water course. They will be monitored for a number of years, and will provide information on water quality improvements associated with Clean Water Atlanta.

5. CSO Consent Decree Compliance

The City's Combined Sewer Overflow (CSO) Remediation Plan which must be completely implemented by November 2007, will enable the City to achieve the highest water quality at the lowest cost within the shortest time frame. A combination of technologies will be employed:

- Sewer separation of a portion of the combined sewer area achieving a 90% separation of the total sewer system and eliminating the need for two CSO control facilities.
- Tunnel storage and treatment system to capture and treat 99% of the sewage and 85% of stormwater from remaining combined area.

This will reduce the incidence of CSO events from 60+ per year at existing CSO facilities to an average of four per year at the four remaining CSO facilities. These remaining overflows will be screened, disinfected and dechlorinated before discharge to a receiving stream.

Why Trenchless?

The City is no stranger to extensive utilization of trenchless methods. Over 200 miles of small diameter sewers have been rehabilitated to date for the most part utilizing cured-in-place pipe. The City anticipates escalating its rehabilitation program to an average annual expenditure of $100 million over the next 8 years, expanding its arsenal of rehabilitation methods including pipe bursting and internal local repair sleeves.

With concern for substantial impact to its citizenry and visitors during implementation of an estimated 667 miles of rehabilitation, and escalating cost of traditional repair and replacement methods, trenchless methods for achieving City objectives make much sense.

The Atlanta "Story" Continues

A regulatory mandate set the stage for the City to develop a responsive, comprehensive plan, namely, the Clean Water Atlanta initiative with its Five Point Plan. Several pivotal work horses had to be rapidly engaged for heavy lifting duties to ensure the City's success including minimizing any further penalties and fines. The subsequent papers in this track describe these efforts, and their results and lessons learned.

SSES and Rehab Selection Process (Paper Two)

This paper discusses the process involved in synthesizing the huge volume of data arising from the SSES study through an in-depth QA/QC process. It ensures that the data is complete, accurate, reasonable and exhibits full connectivity prior to upload to the GIS Hub (Paper Three) and before the Hydraulic Modeling Group (Paper Four) engages in performance of analyses in support of the rehabilitation selection process.

The GIS Enterprise Hub (Paper Three)

This paper provides a description of how an enterprise-wide GIS system was developed for the Department of Watershed Management, Wastewater Division, whereby GIS information could be accessed/distributed via the Internet using conventional web-based tools and ESRI's ArcIMS (Internet Mapping Service). This tool is essential to the execution of SSES, hydraulic analyses and sewer rehab method selection.

Hydraulic Modeling (Paper Four)

Finally, this paper presents the City's hydraulic modeling approach, decisions in software selection and modeling criteria selection, results of analyses, and lessons learned.

Acknowledgements

Compliance with all terms and conditions of the Consent Decree and First Amended Consent Decree are the responsibility of the City of Atlanta, Department of Watershed Management under the leadership and direction of Robert J. Hunter, Commissioner. William Sukenik is the Senior Deputy Program Manager of the department's Program Management Team, and a principal engineer with MWH Americas, Inc., Atlanta, Ga.

References

United States District Court Northern District of Georgia, Atlanta Division, Civil Action File No. 1:98-CV-1956-TWT (1999)

United States District Court Northern District of Georgia, Atlanta Division, Civil Action File No. 1:95-CV-2550-TWT (1998)

Atlanta Track – Paper Two

Atlanta's SSES & Integrated Sewer Rehabilitation Selection Process'

R. E Hutchinson[1], C. Eng., H. K. El-Sayegh[2], Ph.D. and L. Chambers, P.E.[3]

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Abstract

Paper One noted, along with several other requirements, that the FACD includes a task to evaluate and rehabilitate the City of Atlanta's sanitary system. This paper – Paper Two – provides an overview of the City's approach to Sewer System Evaluation Surveys (SSES) and the Rehabilitation Selection process within the terms of the consent decrees.

The mission of the SSES and Rehabilitation Selection Task is to establish a comprehensive and accurate inventory of the assets of the City's wastewater system; to identify sources of inflow and infiltration within the system and to perform a condition assessment of wastewater system assets, incorporating sewers, manholes and laterals. The City's wastewater system has been conveniently split into six sewer groups for the purposes of demarcating areas of the City to be studied over the 14 year period of the FACD. With the findings from this study an appropriate prioritized list is scheduled of sewers, manholes and laterals identified for remedial action. This procedure will be repeated for each of the six sewer groups in turn with the intention of implementing the work in a timely, low impact and cost effective manner. Reporting requirements include the submission of a report to EPA/EPD at the conclusion of each of the six SSES studies.

Although not a prerequisite of the FACD nevertheless a decision was taken by the City of Atlanta at the time the consent decrees were signed in 1999, that in order to address the widespread problem of spills – which, as outlined in Paper One, amounts to over 550 SSOs (unpermitted discharges and spills to dry land) annually – that the entire public sewer system would be targeted for evaluation, i.e., 100% internal sewer condition assessment and 100% internal manhole condition assessment.

The paper discusses the process involved in synthesizing the huge volume of data arising from the SSES study through an in-depth QA/QC process ensuring that the data is complete, accurate, reasonable and exhibits full connectivity prior to upload to the GIS hub – see Paper Three - before being relayed in turn to the Hydraulic Modeling Group – see Paper Four - for micro-modeling analysis prior to the commencement of the Rehab Selection Process.

The Rehabilitation Selection Process, using a specially developed GIS web-based tool, is reviewed in the paper. The tool integrates survey data, rehabilitation methodologies, sewer hydraulic modeling data and associated rehabilitation cost estimates. This allows program design engineers to efficiently determine suitable rehabilitation designs for each element of the system. To date, the tool has been used to generate sewer and manhole rehabilitation designs for just over 10% of the system. For the portion of the system evaluated thus far, various trenchless technologies have been identified as the appropriate solution for a significant percentage of the rehabilitation. High utilization of trenchless technologies is targeted for use throughout the program to provide cost effective system upgrade and at the same time minimal disruption to the community.

Introduction

Paper One noted, along with several other requirements, that the FACD includes a task to evaluate and rehabilitate the City of Atlanta's sanitary system. This paper – Paper Two – provides an overview of the City's approach to Sewer System Evaluation Surveys (SSES) and the Rehabilitation Selection Process within the terms of the consent decrees.

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Although not a prerequisite of the FACD nevertheless a decision was made by the City of Atlanta at the time the consent decrees were signed in 1999, that in order to address the 600 spills experienced annually throughout the City, the entire public sewer system would be targeted for evaluation, i.e., 100% internal sewer condition assessment and 100% internal manhole condition assessment.

The Rehabilitation Selection Process, using a specially developed GIS web-based tool, is reviewed in the paper. The tool integrates survey data, rehabilitation methodologies, sewer hydraulic modeling data and associated rehabilitation cost estimates. This allows program design engineers to efficiently determine suitable rehabilitation designs for each element of the system. To date, the tool has been used to generate sewer and manhole rehabilitation designs for approximately 20% of the system. For the portion of the system evaluated thus far, various trenchless technologies have been identified as the appropriate solution for a significant percentage of the rehabilitation. High utilization of trenchless technologies is targeted for use throughout the program to provide cost effective system upgrade and at the same time minimal disruption to the community.

Background including Prioritization of Work

As noted above, the City's wastewater system was divided into six Sewer Groups in the FACD. Table 1 below provides an overview of the City's wastewater system attributes by Sewer Group with the number of sewersheds in each Sewer Group. (Note that a sewershed consists of a discrete system of sewers with 10,000 to 50,000 feet of mainline sewer). Not included in Table 1 is the total length of lateral sewer within the public domain extending from the main line sewer to the edge of the public right of way amounting to an additional 246 miles of sewer.

Sewer Group	Number of Sewersheds	Length of Sewer (miles)	Original % of Total
One	52	331*[3]	20.0
Two	51	366*[3]	22.2
Three	50	302	18.2
Four	45	255	15.4
Five*[1]	35	226*[2]	13.7
Six*[1]	27	174*[2]	10.5
Total	260	1654	100

*[1]Combined Sewer Area. *[2]Includes approx. 100 miles of separate sewers *[3]Complete

Table 1 – City Sanitary Sewer System

Figure 1 illustrates the relationship between the various Sewer Basins, Sewer Groups and Consent Decree dates for completion of all SSES and Rehabilitation work. It is noteworthy that each Sewer Group consists of individual as well as clusters of sewersheds that are more or less distributed throughout the City and are not Sewer-Basin-centric. Ostensibly Sewer Group One contained the worst sewers within the terms of the selection criteria outlined below and because the impact of rainfall dependent infiltration/inflow (RDI/I) was the least in the combined areas, this was chosen last in priority, i.e., Sewer Groups Five and Six, for the overall study.

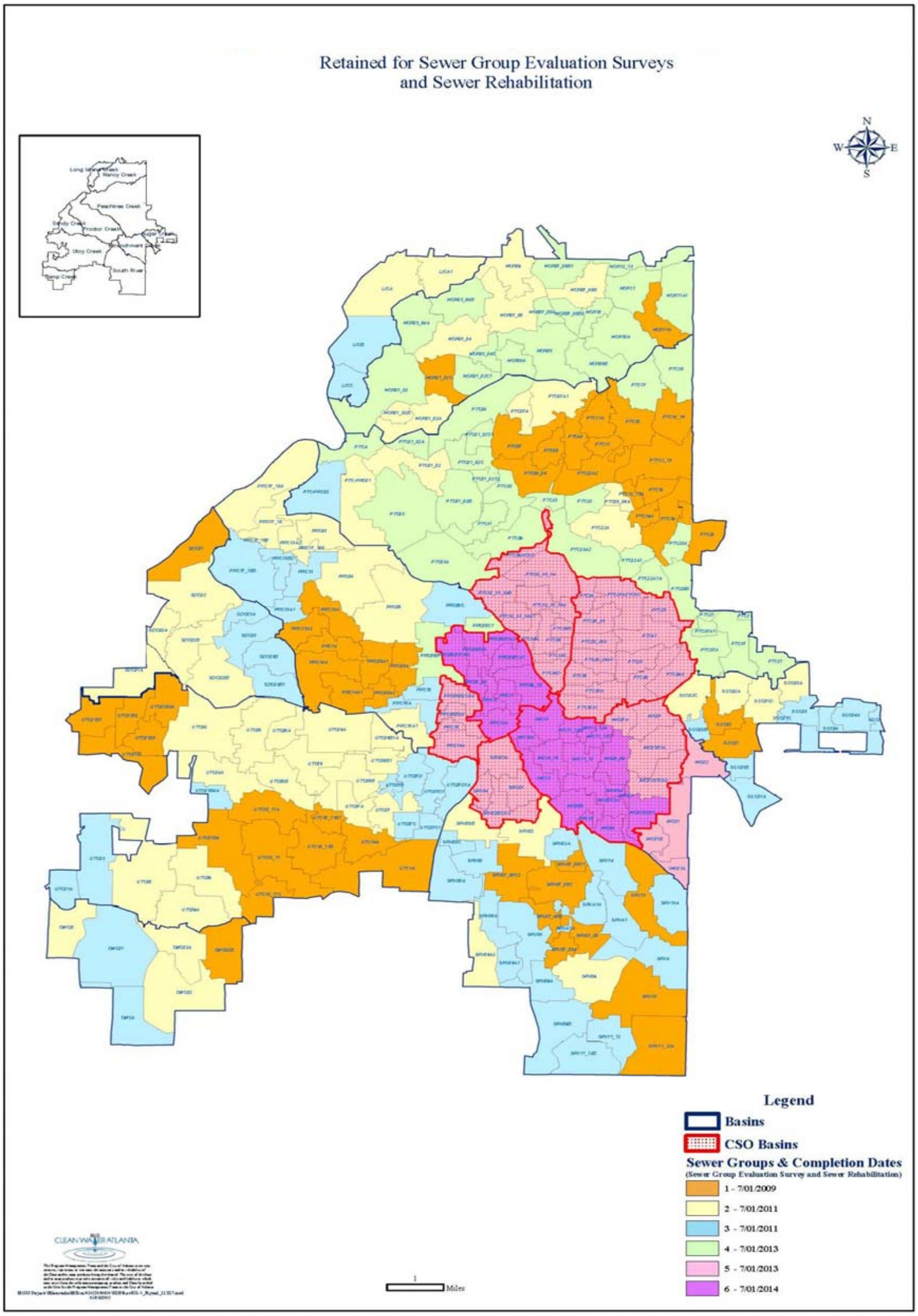

Figure 1 – Original Grouping of Sewersheds

The criteria for the initial prioritization of the Sewer Groups took account of ten items that related to 1) Frequency of Overflows, 2) Severity of Rainfall Dependent Infiltration/Inflow (RDI/I), 3) Risk to Surface Waters (Creeks), 4) Impact Failure, 5) Status of Ongoing Rehabilitation or Renewal, 6) Available Capacity in Sewers, 7) Judgement of Sewer Operation & Maintenance Division, 8) Relative Impact of RDI/I from Jurisdiction Outside the City's Control, 9) Proposed Development Intensity and 10) Location of Sewer Within the Combined System.

SSES Inspection Activities

SSES investigative activities include:

- Sewer inspection using CCTV, sonar and TISCIT (combined CCTV/Sonar inspection),
- Manhole location/inspection and GPS manhole positioning activities,
- Smoke and dye testing,
- Building plumbing location/inspection,
- Flow isolation (plugged segments of sewer utilizing a downstream weir to determine infiltration between 2.00 a.m. and 5.00 a.m.),
- Lateral inspection using a flexi-camera or robotically launched TV camera or testing (with smoke), and
- Temporary flow monitoring, and hydrological data acquisition (supplemental rain gauge monitoring).

In addition to the standard SSES activities, each SSES contract encompasses limited "construction" elements including:

- Locating, uncovering, and raising manholes to grade
- Limited cleaning of manholes and sewers
- Identifying and improving sewer access points (including construction of access roads to the public sewer from the public right of way), and
- Performing point repairs to sewers that urgently require a structural remedy in the interests of public safety and continuous service performance

At this time with the SSES studies for Sewer Group One and Two complete, over 45% of the City's system has been surveyed. The overall SSES schedule indicates a staggered completion of each of the remaining Sewer Groups with full fieldwork completion in 3rd Quarter 2011. Currently, rehabilitation activities for Sewer Group One are proceeding while design of Sewer Group Two Rehabilitation is currently underway.

Following completion of Sewer Groups One and Two SSES work, approximately 15% more footage and over 25% more manholes have been added to the overall inventory of City of Atlanta wastewater assets.

Submission of SSES Data by the Contractor

Detailed specifications are given to the SSES contractor for the extent and format of data to be submitted for the various forms of SSES activity, noted above, within the

SSES contract documents. Because up to four contractors are used at any one time specific training in the various data reports including defect coding has been given by the City of Atlanta. Data requirements included at least 85% in logged data and 95% accuracy in header data.

Primavera Expedition is used to track the large quantity of submittals received from the contractor. Expedition is also used to assist in processing pay applications, store inspectors daily diaries and configure minutes of meetings.

Types of data received include data files, videos and corrections for Sewer Condition Assessment (CCTV etc.) activities; and databases, forms, photos, as built maps, and corrections for Manhole Condition Assessment activities. Similarly, databases, forms, photos (where appropriate) and corrections are received for Smoke Testing, Dye Testing and GPS activities accordingly.

In addition to the data submittals from the contractor, 'I/I Credit' forms (a credit system that allows development to proceed in areas of the city which are capacity limited through the construction of point repairs and other rehabilitation initiatives that reduce I/I), 'Right of Entry' forms, and 'As-built' maps are also submitted.

Typical Field Inspection Life Cycle

On receipt of a typical data set from an SSES contractor a typical data review cycle includes both manual and digital review of data, compilation of error reports for both manual and digital errors, submission of error report back to the contractor, receipt of corrections from the contactor followed by further review and a repeat cycle or cycles as necessary to correct deficiencies. A data set generally is submitted in the context of a sewershed. After a 'Test' sewershed a contractor is allowed to proceed with four sewersheds and only on the completion of an SSES activity, e.g., internal condition assessment of all sewers within the four sewersheds, is the contractor allowed to proceed to the next sewershed to proceed with internal condition assessment. The contractor is encouraged to submit manhole data first to ensure that the connectivity between manholes corresponds and so that annotation of the CCTV video when carried out will also correspond.

Typical manual review of data involves checks for reasonableness, accuracy and completeness of such things as structural defects, service defects, construction feature reporting, picture quality and annotation of initial video frame per segmental length, as well as appropriateness of running measurements.

Digital data review involves checks for connectivity of data (manhole reference numbering, sewer size, sewer material, sewer length and sewer depth), possible reverse slopes, validity of codes, completeness of data, discrepancies between CCTV and GPS lengths, manhole invert errors (invert of incoming pipe lower than invert of outgoing pipe), GIS correlation check (plot of GPS positioning of all manholes within a sewershed), deviation of manhole dimensional configuration, versus standard dimensional configuration and sewage flow direction check.

The ultimate test of the data within a sewershed is the merger of all the data before uploading to GIS so that Rehab Design, Modeling and Mapping can proceed.

Closing Sewersheds

Following all the above checks and establishment of completeness the data, the SSES data manager from the SSES team uploads the data to Oracle and notifies the GIS team manager. The GIS team run QA/QC checks and identifies incompatibilities, if any, with GIS data. The significance of the problem is reviewed and resolved between the two teams where problems exist. On successful completion of the upload the GIS team manager notifies the Modeling team manager that data is ready and exports data into a the format required by the Modeling team. Figure 2 illustrates the typical flow of activities from receipt of raw SSES data from the field (SSES contractor) to final acceptance and release of the next sewershed.

Rehabilitation Design

One of the main challenges that face rehabilitation decision makers is the utilization of all the collected field data in determining the best approach to sewer system improvements. The shear volume of data and the variety of sources makes it a considerable undertaking. Data sources include GIS, CCTV videos, smoke testing photos, manhole inspection reports, spill reports, hydraulic modling etc. Additionally, there are several rehabilitation methodologies that may be used to remedy the defective sewers. Each of these methods has various elements associated with it including, but not limited to, by-pass pumping, lateral reinstatement, and site restoration.

Given all the elements that contribute to the rehabilitation design process, it is imperative that the rehab designer has a holistic approach that provides access to all data sources in order to identify a cost effective rehabilitation solution.

To facilitate an integrated rehabilitation design system, Program Management Team, (PMT) staff developed the Rehabilitation Selection Tool (RST). It is a GIS web-based application that integrates all SSES and GIS data. Several proven rehabilitation methodologies are incorporated into RST along with the respective bid items.

Also critical to the process is the cost matrices containing the unit costs of the bid items. These costs are used to calculate the engineer's estimate for the proposed rehabilitation solutions. The unit costs are based on recent bid information and are updated every six months or whenever there is reason to suggest considerable changes in prices (e.g., surge in fuel costs).

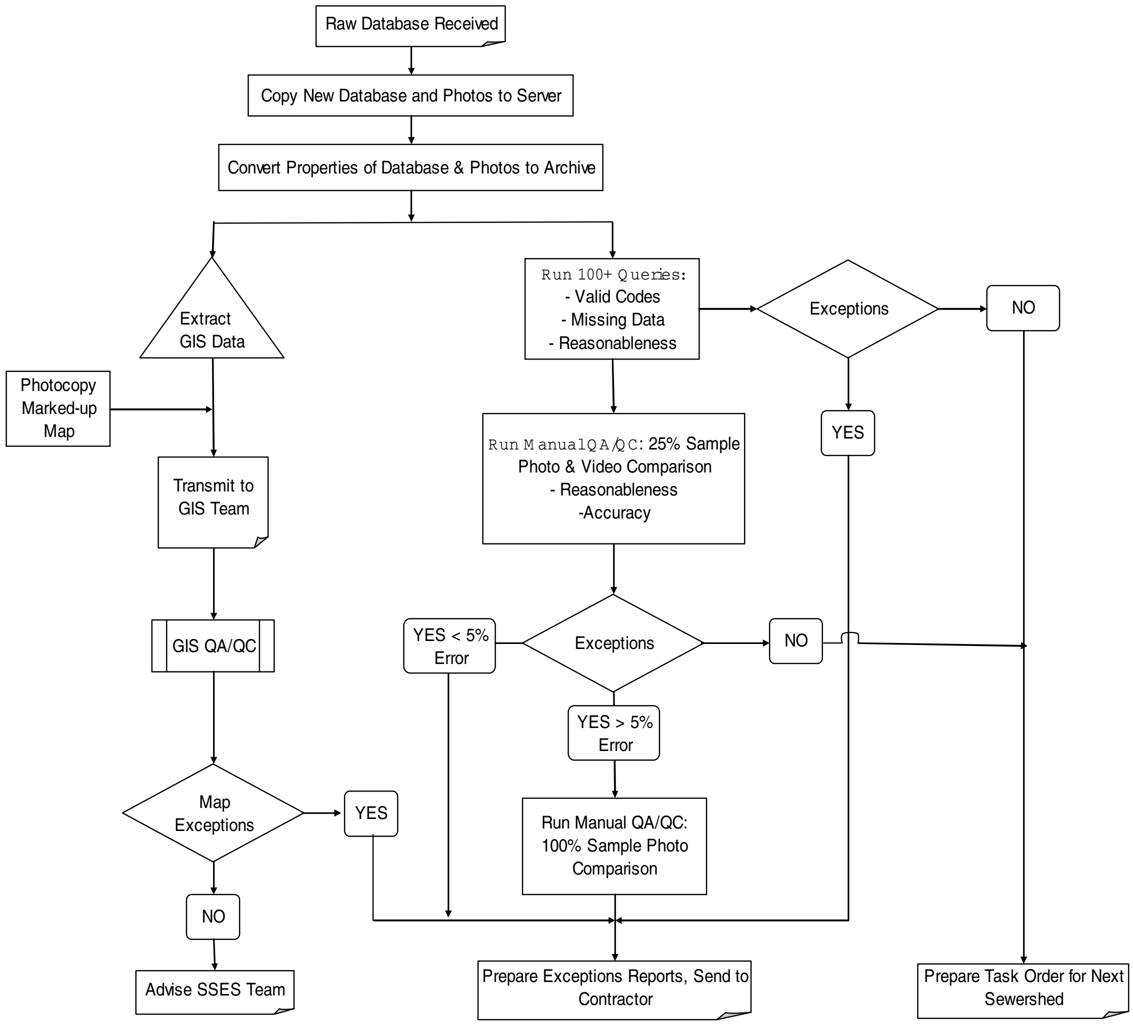

Figure 2 – Digital Data Review Flow Chart Per Sewershed

Preparation of Sewershed Design Binders

Upon completion and verification of SSES data for a given sewershed, these data are uploaded to the GIS database and made available for use by all PMT staff. Sewershed design binders are then prepared by SSES staff as a submittal to the Rehabilitation Design Team. These binders contain data reports showing the assets in each sewershed along with pertinent information including CCTV footage location, documented spills, and point repairs completed during SSES work. The design binders are used to track the status of the rehabilitation design for each sewershed and whether or not some of the required rehabilitation has been completed during SSES work. The spills records included in these binders also assist the designer in locating problems not identified by SSES such that a comprehensive and pertinent solution is selected.

Accessing the Rehabilitation Selection Tool

Once the design engineer receives the sewershed binder the design process commences by accessing the username and password secured RST. Within RST the engineer can zoom to any portion of Atlanta's sewer system to identify and examine a specific sewer segment. Alternatively, the specific segment may be selected by inputting its ID. A toggle button allows the user to switch between the displays of structural and service defects in two separate views.

Figure 3 shows a snapshot of the structural defect view for a sewer in sewershed UTC06. Each specific defect is listed along with the distance from the upstream manhole, start and end clock position, defect grade, and comments.

As illustrated in Figure 3 the designer has access to the source of data, and hence can choose to view the CCTV footage (where TV is indicated) or the photos (where VI for visual or ST for Smoke Testing is indicated) in order to verify the extent and severity of defects of interest.

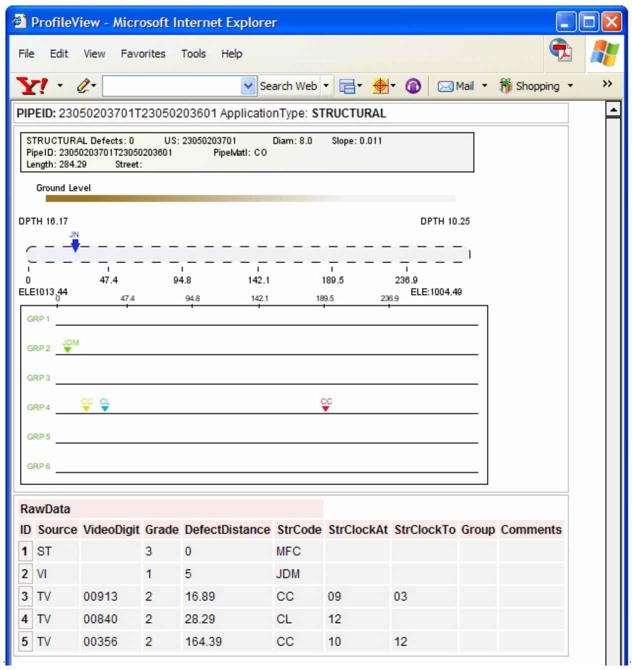

Figure 3 – Structural Defects View from the Rehabilitation Selection Tool

Rehabilitation Selection Guidelines and Approach

Based on the condition assessment, the designer determines the main rehabilitation approach according to the guidelines developed by the PMT. These guidelines identify the approach to rehabilitation using RST. The main rehabilitation selection is based on the presence and frequency of a group of essential *structural* defects (e.g. breaks, holes, multiple fractures, deformations, and large joint displacements). Extensive *service* defects such as multiple root masses and root taps may also warrant rehabilitation action.

RST has a modular structure where the designer chooses from the following four main categories:

- Capacity Relief Projects
- Rehabilitation
- Capacity assessment, Maintenance, Operation & Management (CMOM) and Deferral
- No Rehabilitation

Choosing any one of the above categories will channel all further actions to that particular category. The 'Capacity Relief Projects' category is specified for all trunks and large sewers (greater than 24-inch diameter) designated for upsize and/or slope change. 'Rehabilitation' is the main category and is specified for trenchless rehabilitation as well as traditional open cut replacement methods. This category also includes root removal, cleaning and manhole rehabilitation and replacement. Sewers for which action is "deferred" and sewers that need to be "monitored" for a period of time before a final rehabilitation decision is to be made (if appropriate), are placed in the CMOM and Deferral category. In cases where only minor defects or no defects are present, the sewers are placed in the 'No Rehabilitation' category

Figure 4 shows the lining selection screen as a subset of the rehabilitation category. As illustrated, a drop-down list provides multiple selections for the type of lining – in the example shown it is CIPP. The user also chooses traffic control and by-pass pumping requirements, in addition to other items that may be associated with the lining process. Unit costs for the main rehabilitation and associated work are obtained from the cost matrices to calculate the total cost for rehabilitating the sewer segment.

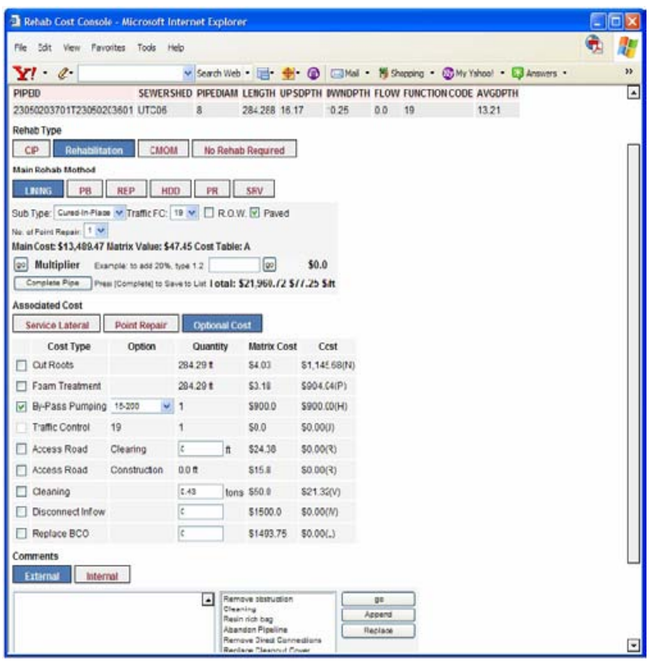

Figure 4 – Structural Rehabilitation View from the Rehabilitation Selection Tool

Although RST has a detailed approach, the RST process does not eliminate the need for engineering judgment. All rehabilitation selections are coordinated with capacity relief and modeling recommendations. The recommendations are evaluated and some of them are incorporated into the rehabilitation work (e.g. upsizing from 8-inch to 18-inch diameter), while other recommendations are deferred to capacity relief projects (e.g. upsizing from 21-inch diameter and above). The designer facilitates expedient solutions by making every effort to adopt the same rehabilitation methodology for adjacent sewer segments and thus eliminate situations in which rehabilitation could alternate between pipe bursting and CIPP along specific reaches of sewer. (Mobilization costs and disturbances to the general public are mitigated accordingly).

A detailed QA/QC process follows the completion of rehabilitation design. Database queries are used to generate error and warning reports. These reports are used to verify that the design guidelines have been followed. This process is also used to avoid issues caused by SSES anomalies, e.g. duplicate defects need to be addressed only once.

Constructability Reviews and Easement Research

Following the data QA/QC process, field constructability reviews are performed for each sewershed. During these field reviews PMT staff evaluates the actual field conditions, which might not be clear from the aerial photography. This includes features such as electric poles, fire hydrants, and patios or decks. Other conditions might have changed since the survey was performed such as newly paved streets or recently landscaped plots.

Another important element in the Rehabilitation Selection Process is easement research. All sewers intersecting private property within a sewershed are identified and submitted for easement research. In some cases sewers pass under structures – homes and other buildings - and are given special attention if rehabilitation is required. Depending on the recommended rehabilitation methodology, it is determined whether land acquisition is necessary or not. Upon completion of the constructability reviews and easement research, bid documents are prepared for procurement. The procurement process takes approximately four to six months from advertisement to issuance of the Notice to Proceed (NTP).

At the time of writing this manuscript, rehabilitation design for Sewer Group One is complete, and the design for the first phase (24 sewersheds) of Sewer Group Two has just commenced. It is expected that this phase will be completed by the summer of 2007, with work on the second phase to follow immediately.

Rehabilitation Contracts

Rehabilitation contracts are divided into two main categories. Approximately 75% of the work is performed under 'Defined Contracts', the first category, with specific quantities listed for each bid item. Contractors are expected to implement the designs produced by the PMT with minor variations. The second category, 'Undefined Contracts', covers work for the remaining 25% of the work. Unit prices are supplied by contractors for estimated quantities that are representative only. Work is then performed under close Construction Management Supervision and based on capacity credit requirements.

'Defined Contracts' are configured to have both location and rehabilitation method specificity. 'Undefined (Annual) Contracts' are configured to have rehabilitation method specificity only. There are five geographic areas in the city, namely north, south, east, west and central.

Current Rehabilitation Design and Construction Status

To date approximately 20% of the total sewer system length has been analyzed and rehabilitation assignments have been made to approximately 43% of that length. The first phase of construction (approximately $37 million) is currently in progress and is expected to be completed in 2008. At the time of writing this paper, the second phase is in the final procurement process.

Conclusions

The considerable amount of data collected during the SSES and the variety of data sources necessitates the use of an integrated approach to sewer system rehabilitation.

The comprehensive QA/QC review of SSES data outlined above is imperative to the successful upload of data and fulfillment of the specified intent of the contract specifications, GIS map production, and modeling needs. The process enables contractors to price and plan work competitively and with certainty as well as subsequently implement work expeditiously against GIS derived maps that provide a near perfect documentation of the assets in the field.

By incorporating all SSES and rehabilitation methodology data in a web-based GIS application, the designer is able to make a quick, informed decision on the rehabilitation. Constructability field reviews and easement research that follow help evaluate field conditions and make adjustment to the design or construction approach. Construction services are then procured for multiple contracts based on the geographical area and rehabilitation methodologies. This integrated approach fosters a competitive bidding process by providing accurate quantities and portrayal of field conditions.

Atlanta Track – Paper Three
Clean Water Atlanta Enterprise GIS

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In 1998 and 1999, the United States Environmental Protection Agency (EPA) and Georgia Environmental Protection Division (EPD) completed settlement negotiations associated with two complaints against the City of Atlanta for various violations of the United States Clean Water Act and Georgia Water Quality Act. This resulted in the recording of the Consent Decree (CD) related to Combined Sewer Overflows (CSOs) and the First Amended Consent Decree (FACD) associated with Sanitary Sewer Overflows (SSOs). They mandated improvement of the City wastewater collection and transmission systems including wet-weather control for combined sewers, and rehabilitation and relief of capacity-limited sanitary sewers.

The City wastewater collection and transmission systems include approximately 1900 miles of sewer mains and approximately 37,500 manholes. Fifteen percent of the system is comprised of combined sewers while 85 percent of the system consists of separate sanitary sewers. The FACD simply mandates the development of a mapping system which inventories the sanitary and combined sewerage systems. This paper will describe how the GIS task has gone beyond the requirements of the FACD to build an integrated enterprise database system explaining the benefits of doing so. In addition to GIS information, this integrated enterprise system also includes information pertaining to scheduling, project management and contracts, modeling, capacity permitting and Inflow & Infiltration (I&I), flow monitoring data and work management and planning.

Under the FACD, the mission of the GIS task is to maintain and improve the City's GIS databases pertaining to the sanitary/combined collection system in addition to providing maps and spatial data to contractors, consultants, elected city officials, city employees, and the development community. To provide these services we implemented an enterprise-wide GIS system for the City of Atlanta, Department of Watershed Management, Wastewater Division, whereby GIS information can be accessed/distributed via the Internet using conventional web-based tools and ESRI's ArcIMS (Internet Mapping Service) and ArcGIS Server. The data are stored in an Oracle database and accessed through ESRI's Spatial Database Engine (SDE). The implementation of this system required the conversion of CADD-based sewer/manhole drawings to shapefiles/coverages and finally to a geodatabase format. In doing so, we designed a wastewater object model to best represent the City's collection system. Domains, subtypes, relationship classes and geometric networks all play a part in describing, modeling, and automating procedures to QA/QC the City's collection system.

Specific project activities have included:

- Conversion of CADD-based sewer/manhole drawings to a GIS format
- Updating GIS data with current survey data
- Assisting in QA/QC of the Sanitary Sewer Evaluation Survey (SSES) data and update the GIS with these data when appropriate (see details in Paper Two)
- Developing an object-based data model to accurately represent the City's collection system.
- Uploading and maintaining data in Oracle RDBMS using ArcSDE to access information.
- Creating a GIS application framework to quickly deploy multiple interactive mapping websites for easier access of data and maps.
- Developing an Intranet-based rehabilitation and cost estimate application to aid the City in its mandate to rehabilitate the aging collection system (see Atlanta Track - Paper Two).
- Creating an address verification application in order to develop master address database to include City's dispersant address files used by different City departments.
- Creating automated I&I credit reports by sewershed calculated from CCTV condition information together with completed rehabilitation.
- Managing GIS links to Hydraulic Modeling software, creating tools to assist in the processing of flow monitoring data, developing tools to predict/calculate risk analysis for overflows using 40 year storm data (See Paper Four).
- Developing traffic coordination website to provide the public information about projects in the right-of-way.
- Developing a Work Order Management Website which allows a user to easily query list and display location of Hansen and Maximo Service Requests and Work Orders, run reports and open, change status and close work orders all from one interface.
- Developing real-time mobile applications using ArcGIS Server

The following pages will describe what an enterprise GIS is, what are some of the benefits and what are some of the key features which comprise our enterprise GIS database architecture.

Enterprise GIS Architecture

In recent years many large organizations and municipalities have switched from independent, stand-alone GIS systems to more integrated approaches that share resources and applications. The basic idea of an enterprise GIS is to address the needs of departments collectively instead of individually. The development of one comprehensive infrastructure minimizes potential conflicts and misunderstandings and can result in significant cost savings and performance improvements. [1]

Enterprise GIS is an organization wide approach to GIS implementation, operation, and management. It integrates spatial data and technology across the organization, coupling centralized management with decentralized use. Enterprise GIS involves an integrated database and system architecture that provides users with different types and levels of access and functionality — tailored to support their work processes, locations, and devices. Its design also integrates with other data and systems within an organization. It is based around industry IT standards and web services. Users of spatial data spend more time on the analytical capabilities and business functions of a GIS and less time searching for, compiling, and integrating the data they require.[2] By developing an Enterprise GIS, non-GIS applications and systems can easily access GIS functionality, and GIS applications can easily access functionality and data provided by other systems. For example, many traditional applications, such as I&I credit calculations, rehabilitation conceptual design, hydraulic modeling, can include rich GIS functionality for purposes of investigation, validation, analysis, or mapping. Similarly GIS applications can integrate transactional data for analysis, planning and reporting.[3]

Data Warehouse

Most enterprise GIS applications use some type of geospatial data warehouse that loads information from operational databases into a centrally managed and distributed system. When data is stored in a data warehouse, all users have immediate access to the most accurate and up-to-date version, so there should no longer be problems with departments using outdated information. A warehouse also makes it easier to manage GIS and IT resources. [1]

A comprehensive enterprise GIS should provide a common platform for data collection, storage, authorized and secure access to spatial and aspatial data, harmonize the work flow of respective departments and disseminate information for the benefit of its users at large. In Atlanta, we utilize an Oracle solution for our common platform.[4]

The cornerstone of our Enterprise GIS is the data received from the Sanitary Sewer Evaluation Survey (SSES). The city is conducting a Sanitary Sewer Evaluation Survey (SSES) for all manholes and pipe segments. All manholes (including those covered) will be inspected not only for location, elevation and size but also for condition. All pipe segments will be surveyed for condition (using Closed Circuit TV - CCTV) and connectivity. The SSES contract requires the delivery of data in a stringent database format so that rigorous QA/QC processes can be automated and run prior the data being accepted and the contractor being paid for services. Processes have been developed to ensure correct geographic coordinates, pipe size, slopes, elevations, manhole/ pipe identifiers and connectivity. Once these data have been quality assured, they are compared to the existing GIS data. The GIS data are updated where appropriate. Having an accurate, stable spatial database is the key to an enterprise GIS system being successful. Being that the currency of the data is critical for it to be used effectively on a day-to-day basis, it stresses that this system is continuous rather than static.

After these data have been QA/QC and the GIS data has been updated, all ancillary data concerning the collections systems inspection and condition are uploaded into Oracle. Figure 1 illustrates the database configuration for the City of Atlanta, FACD project.

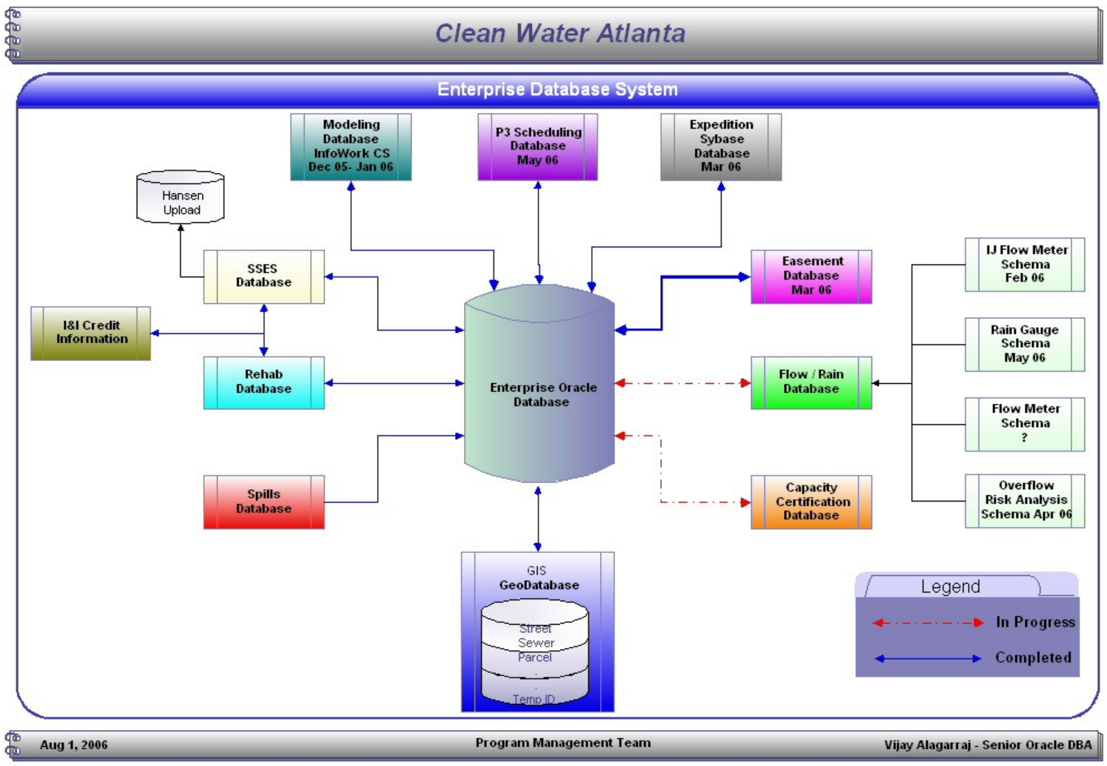

Figure 1 – Database Configuration

Among its many opportunities and benefits, enterprise GIS can streamline work processes, integrate data and systems, reduce duplication of effort and data, enable rapid response to new demands, and best leverage an organization's spatial data and technology investments. It also helps the organization manage and protect its spatial data and technology assets.

Each task have specific requirements for spatial and aspatial data and often, there is an overlap of information crucial to these departments. An Enterprise approach enables access to information across multiple departments, promotes speedy decision-making and transparency in the functioning.

GIS functionality in customized business applications

Enterprise GIS incorporates "location awareness" and "location sensitivity" into front-end applications, back end database computations, field operations, and Web offerings.[3] This approach is exemplified in the 'Rehabilitation Concept Design/Bid Document Tool (Rehab tool). In Figure 2 we can see how the Enterprise Database structure outlined in Figure 1 made the development of this tool possible. The Rehab tool borrows from many of the databases stored in our Oracle solution. The GIS interface is the mechanism which these data are built mainly due to the fact that the GIS interface is a graphical method in which to display masses of information in a manner that a useful and understandable to the end-user.*

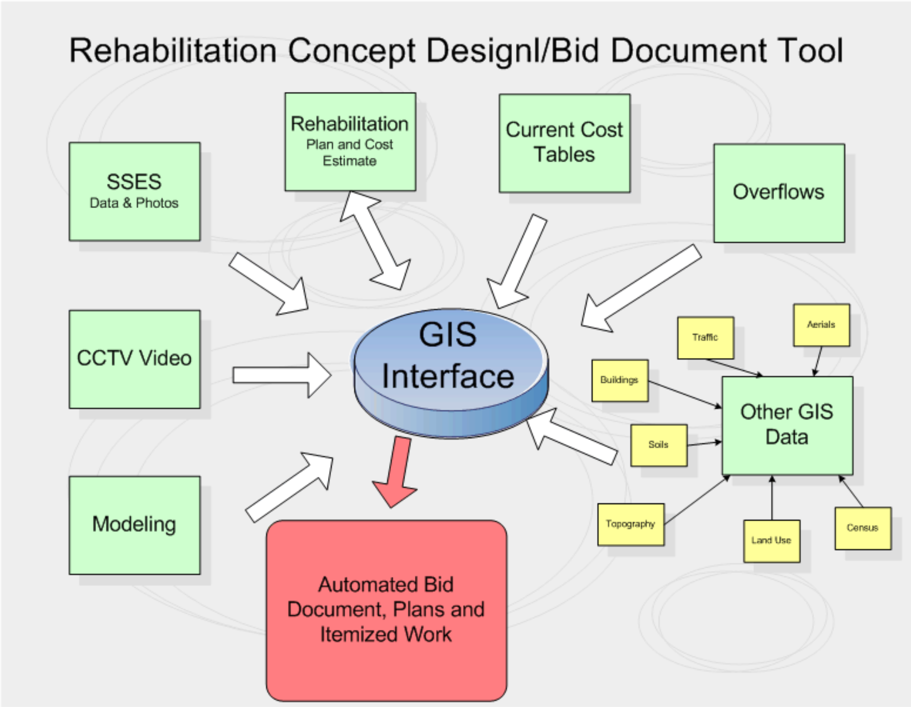

Figure 2 – Rehab Tool

*The rehab tool is more fully described in Paper 4 of the Atlanta Track

An Enterprise GIS delivers both soft and hard benefits - both increasing worker productivity and capacity to deliver goods and services to customers.

- Support for the best tools for any job - each department can use its preferred software and data types, while working from a shared enterprise database
- Increasing productivity - eliminate the time wasted to convert or translate data and eliminate out-of-sync data or errors
- Increasing data accessibility - when data is stored in an enterprise database, all users and key decision makers have quick access to the most accurate and up-to-date data.

- Improving communication among departments - an enterprise system requires that all independent systems communicate quickly and effectively, regardless of data format
- Increasing data security - by storing spatial data in a central database, your organization will maintain secure, high-quality data
- Enhance speed, reliability and uptime
- Enable easy web-based access for data sharing and community participation

Reduced redundancy, improved data sharing, faster response, scalability and greater payback from geospatial investments all make Enterprise GIS the wise choice and best return on investment for any business or government agency.[2]

Data Content/User Management System within GIS Web Application Framework

With an enterprise GIS program, it may be possible to develop a custom application and then make minor revisions to meet the needs of individual departments. The Clean Water Atlanta GIS task has capitalized on this development concept in its GIS Web application (Figure 3).

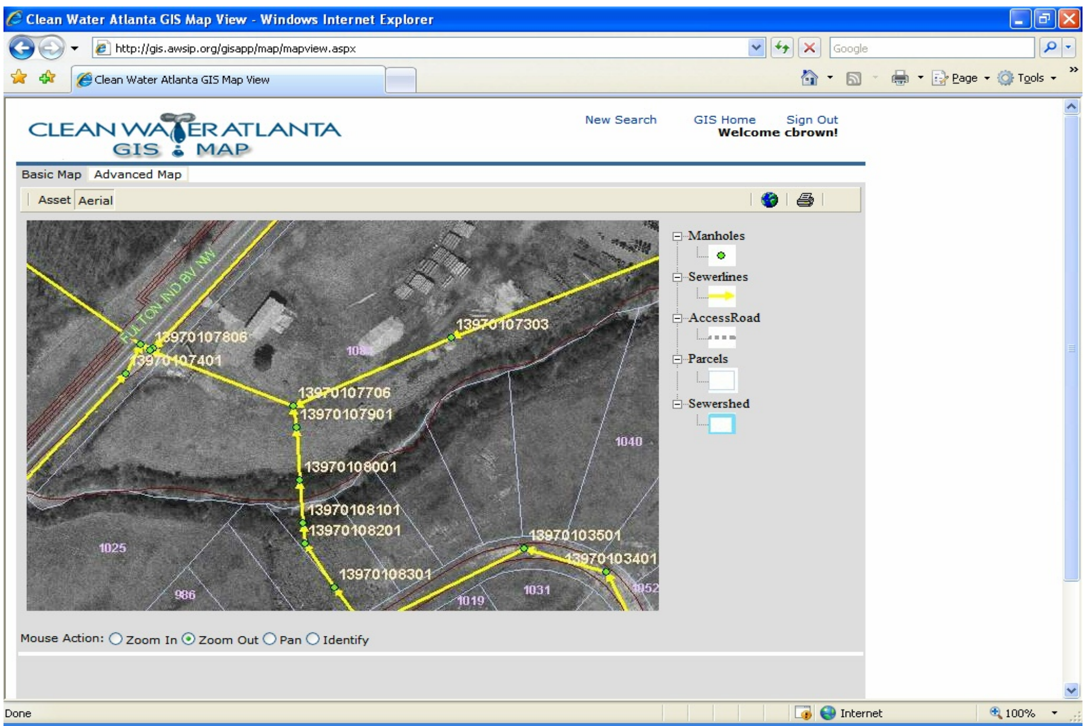

Figure 3 – GIS Website

When working within a large organization, mapping websites need to be designed using a framework which accommodates many users with varying skill sets. In order to accomplish this we designed a framework which allows users from different agencies, departments or user groups to make use of one web application framework but customize their view with their own map content and functionality. The map content and functionality are managed through each user's role and group assignment (Figure 4).

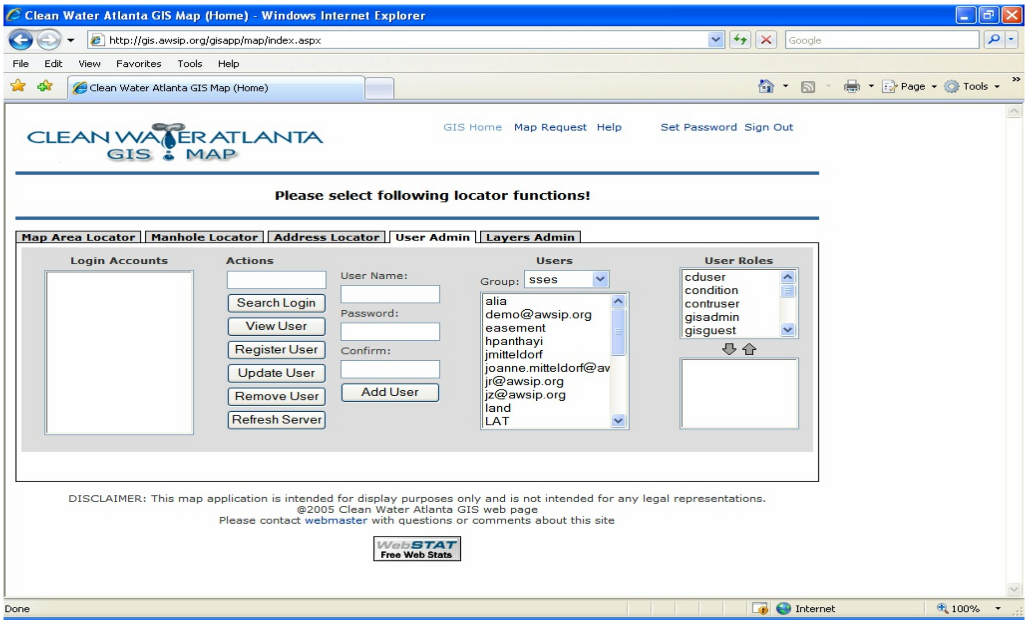

Figure 4 – User Administration Tab

Each group will consist of one person assigned to an administrator role. This person will manage user accounts, map content and functionality for their group through an easy to use, web-based interface (Figure 5). The individual users themselves are given the ability to customize their map view with their own personalized map symbology and save as a template for future use. The application utilizes ArcIMS and ASP.net web service technology, which provides scalability and security.

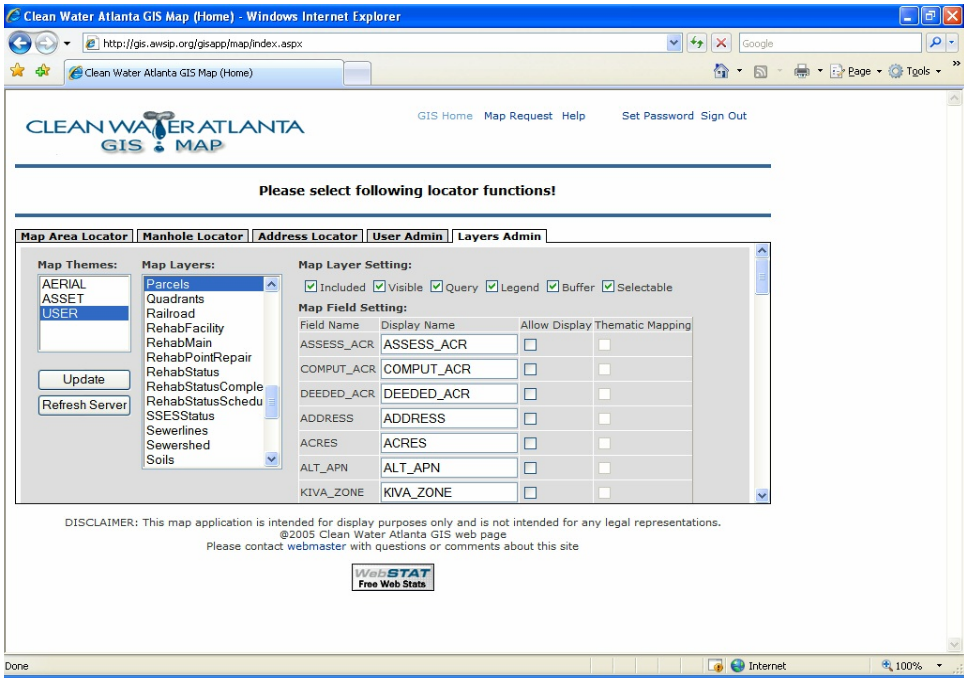

Figure 5 – Content Administration Tab

Conclusion

GIS has historically been a separate, special-purpose resource - useful for some decision making, but isolated within a department or project. As business and government strive to automate business processes and increase productivity, geospatial data and technologies become more valuable when they integrate with other mainstream business information delivery techniques and IT systems.

As a whole the Department of Watershed Management and the City of Atlanta is increasingly adopting an enterprise approach for all their business processes. GIS is an extremely important tool to make use of an enterprise database and allow users to view, analyze and manipulate data in ways they never imagined.

[1] Spies, J.L. (2005, January) Spatial Technologies: Software Strategy: Options for the Enterprise. *Cadalyst*. Retrieved from http://gis.cadalyst.com/gis/article/articleDetail.jsp?id=141055

[2] *Benefits of Enterprise GIS*. (n.d.). Retrieved December 15, 2006 from http://www.fargeo.com/enterprise_gis/index.html

[3] *GIS and IT integration*. (n.d.) Retrieved December 15, 2006 from http://www.fargeo.com/enterprise_gis/it.html

[4] Subash S., Padaki, A. (n.d.) Enterprise GIS for Municipalities – An Integrated Approach. GIS Development. Retrieved December 15, 2006 from http://www.gisdevelopment.net/application/lis/overview/mi03214.htm

Atlanta Track – Paper Four

Hydraulic Modeling – A Tool for Addressing the Consent Decree

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Abstract

Prior to 1915, the 19 square miles that comprised the City of Atlanta, and what is now the core downtown area, utilized a combined sewer system. As the City grew to its present total area of 132 square miles, the trend changed to a separate storm and sanitary collection system. Currently, the City is served by approximately 1900 miles of sewer mains, four wastewater treatment plants, six CSO control facilities, and one CSO treatment facility. The entirety of this collection system is subject to the mandates set by the SSO First Amended Consent Decree (FACD) and the CSO Consent Decree (CD).

The CD and FACD ordered the evaluation and rehabilitation of the City's sewer collection systems through a number of plans. A Prioritization Plan was created at the onset to divide the city's ten sanitary sewer basins into 260 sub-drainage areas (sewersheds). Using the Criterium Decision Software, these sewersheds were then prioritized based on a number of parameters (i.e. structural integrity, capacity, risk to surface water, etc.) and assembled into six sewer groups. The Flow and Rainfall Monitoring Plan, which required both permanent and temporary flow monitors as well as a number of rain gauges, was implemented to gather data for hydraulic model calibration and to identify excessive infiltration and inflow. The Hydraulic Modeling Plan took these components and designated modeling parameters.

Due to the immediate need for a hydraulic model, the City's modeling approach consisted of a three step process. Initially, a macro model was built for only the large trunk sewers followed by a micro model, which added the major outfalls, then the addition of the remainder of the smaller diameter pipes through SSES data – see details in Paper Two – as relayed through the GIS Hub to the Hydraulic Modeling Group – see Paper Three. Calibrated models are used for numerous applications, primarily the development of capacity upgrade proposals, e.g. pipe bursting – see

Paper Two concerning the Rehabilitation Selection Process - as well as capacity relief projects. For this application, a design storm was developed using an analysis of historical rain data to characterize the “typical” storm observed in the City. Running a calibrated model under this design storm identifies system-wide capacity issues. In other applications, many Requests For Information require model analysis to determine cause of overflows, solutions to capacity issues, amounts of infiltration/inflow, and it can also be used to verify the amount of flows attributed to the six inter-jurisdictional municipalities that feed into the City’s system.

In October 2005, the City began to convert all sewer models from XP-SWMM to Infoworks for reasons that included GIS compatibility, enhanced model output reporting, ease of use, stability and reduced run times for very large modeling simulations. Amidst this software change and review of modeling criteria the task is on schedule with currently 954 of the City’s 1600 miles incorporated into the model.

This paper will discuss the hydraulic modeling related CD and FACD requirements, the City’s modeling approach and project’s successes as well as lessons learned.

Introduction

Prior to 1915, the 19 square miles that comprised the City of Atlanta, and what is now the core downtown area, utilized a combined sewer system. As the City grew to its present total area of 132 square miles, the trend changed to a separate storm and sanitary collection system. Currently, the City is served by approximately 1900 miles of sewer mains (15% combined and 85% separated). All sewage flow within the City is treated by three wastewater treatment facilities, six Combined Sewer Overflows (CSO) control facilities, and one CSO treatment facility to serve a population of 1.6 million with a total average daily flow of 182 MGD. In addition to the flow produced within the City, there are six adjoining jurisdictions contributing flow to the City of Atlanta sewer system that comprise 45% of the total flow as shown in Figure 1.

The City of Atlanta is divided into 260 sewersheds that join to form ten sewer basins. As defined by the First Amended Consent Decree (FACD) a sewerbasin contains all portions of the wastewater collection and transmission systems tributary to a trunk sewer entering a wastewater treatment facility (WWTF). The following ten sewer basins are separate from each other, but some of them are hydraulically-linked at the treatment plant:

1. Camp Creek (CMC)
2. Intrenchment Creek (INC)
3. Long Island (LIC)
4. Nancy Creek (NCR)
5. Peachtree Creek (PTC)
6. Proctor Creek (PRC)
7. Sandy Creek (SDC)
8. South River (SRV)
9. Sugar Creek (SGC)
10. Utoy Creek (UTC)

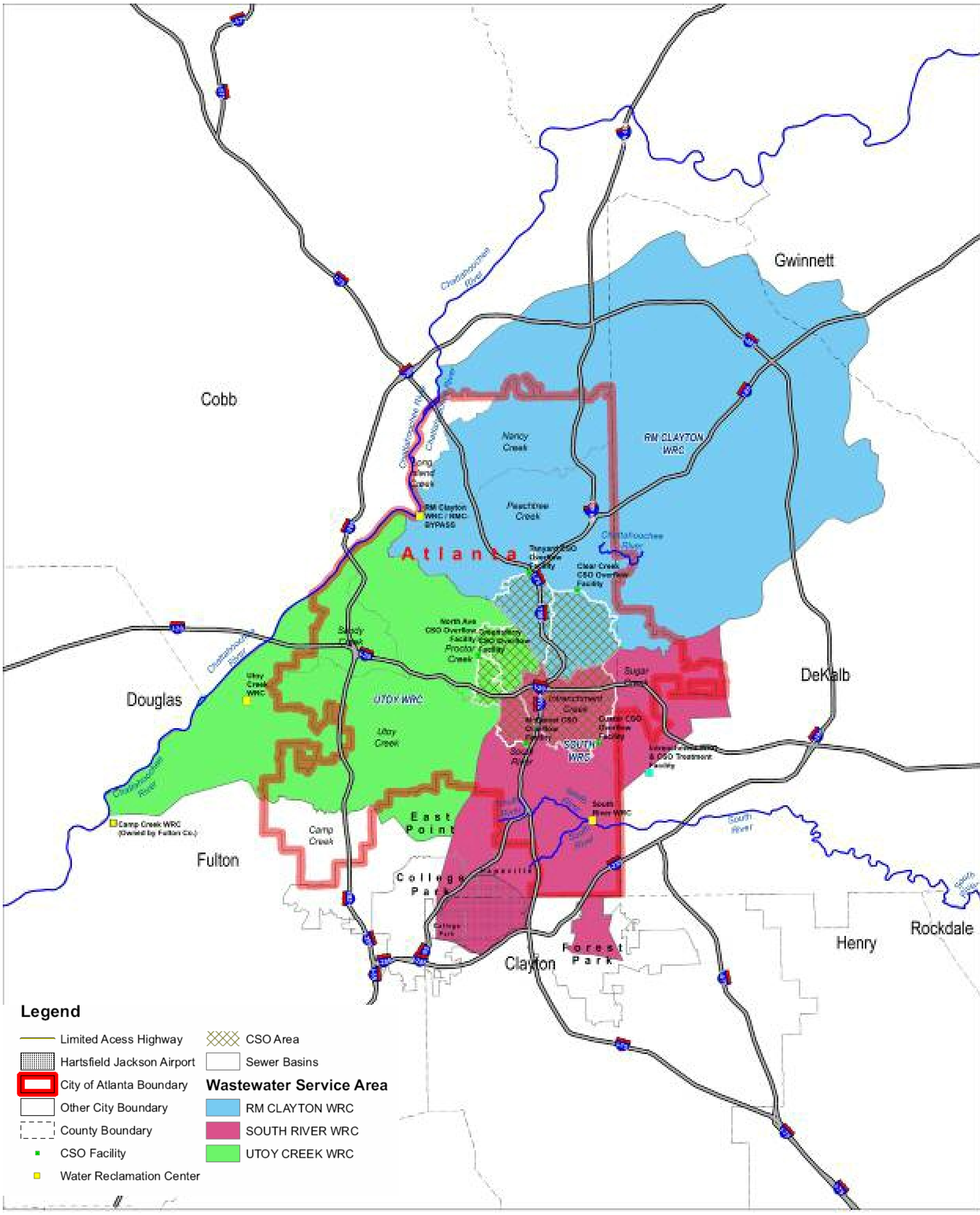

Figure 1 - Wastewater Service Areas

Consent Decree Root

The FACD mandates the elimination of all unpermitted discharges and sanitary sewer overflows to achieve compliance with NPDES permits. This is accomplished through a number of plans under sections VIII.C.1, VIII.C.2. and VII.B.8.D.

One plan is the System-wide Flow and Rainfall Monitoring Plan whose purpose is identifying excessive infiltration and inflow (I/I), quantifying peak flow factors, establishing sewer group priorities, and serves as a data source for hydraulic and hydrologic modeling. The City has submitted this plan to EPA/EPD.

To fulfill the requirements of the FACD, ADS Environmental Services was contracted in 1999 to install and maintain flow meters and rain gauges, process and analyze the meter data and prepare monthly reports. Since implementation, 125 permanent meters and over 500 temporary meters were installed along major trunks and outfalls to establish dry and wet weather flow rates, determine existing capacity within each monitored sewer, and allow the City to determine infiltration/inflow (I/I) contributions to sewer flows.

There are two types of flow meters installed in the system, permanent meters (ADS model 4000) which are primarily located on the trunk lines and temporary meters (ADS model 1502) which are located on the outfalls. The meters measure depth of flow and velocity, from these two parameters flow rate is calculated. Depth is measured by an ultrasonic depth sensor mounted at the top of the pipe and a pressure depth sensor mounted at the pipe invert to take redundant measurements. The velocity sensor in the invert measures the peak velocity then calculates the average velocity from a velocity profile performed in the field. If the flow is contained in the pipe, a cross-sectional wetted area is determined and the flow is calculated using the continuity equation, $Q = A \times V$. If the pipe is in a surcharged state, flow is calculated using the full depth of pipe.

Thirty-two rain gauges (tipping bucket) were installed throughout the City to provide a comprehensive network for establishing a rainfall record. Though not specified in the FACD, the City also utilizes it's existing monitoring equipment as a supplement to the ADS network of meters. The City maintains an additional 103 permanent flow meters and 13 rain gauges. Typically, these flow meters are located along the border of the City and provide information regarding gravity flows entering the City's system (points of entry) from the surrounding interjurisdictional (IJ) municipalities and flows leaving the City's system.

The need for a comprehensive hydraulic model was realized early in the consent decree process through the requirement of a System-wide Hydraulic Modeling Plan. The model was deemed essential to assist in the development of operational procedures that may optimize system capacity, as well as to evaluate the impact of proposed system improvements. This plan is responsible for selecting the modeling software to be used and establishing the modeling approach, data verification, calibration methodology, performance and overall sensitivity.

Modeling Software Selection

In choosing a modeling software for the City, the Hydraulic Modeling Plan evaluated four separate packages: Hydroworks, Mouse, Visual Hydro and XP-SWMM. XP-SWMM was chosen for numerous reasons:

- Compatibility with existing City models built in XP-SWMM.
- Simplicity of the existing City model (i.e. small number of ancillary structures) didn't necessitate the advance ancillary structure capabilities of Hydroworks or Mouse.
- Consistent and stable software support, which can become an issue with the relatively new software Visual Hydro.

As with other modeling software packages, XP-SWMM dynamically simulates the flow within a sewer system by iteratively calculating flows in each pipe segment until it converges on a solution at a predetermined tolerance. As all models, it is heavily dependant on the physical characteristics of the system including pipe diameter, length and slope, network connectivity, manhole coordinates, inverts and ground elevations, as well as facility configurations.

Modeling Approach

Due to the immediate need for a hydraulic model, the City's modeling approach consisted of a three step process to obtain the physical asset data. Initially, in 2000, a macro model was built for only the large trunk sewers (i.e. greater than 24-inch in diameter). This basic model provided the City with an initial capacity assessment of the system. Secondly, over the next four years, one sewer group per year, all major outfalls (greater than 12-inch diameter) were added as defined in the Prioritization Plan- noted in Paper Two. Lastly, all sewer lines 8-inch and above are currently being added resulting from SSES work that began in July 2002- as detailed in Paper Two. The end result is a comprehensive model for the City.

Calibration

Once the physical attributes have been received from SSES and a model is built, the model network is loaded to simulate dry weather flow. As population and trade (Commercial/Industrial) flow data is not readily available, an average dry weather flow (ADWF) is obtained from meter data pertaining to the sewershed. The ADWF is then distributed throughout the model and loaded onto manholes. Diurnal patterns that vary according to the time of day were developed and applied to dry weather flows based on flow data gathered from the meters. The diurnal profiles are derived from the flow meter data. Approximately seven days of ADWF are needed for calibration. For dry weather calibrations, a 10% margin of error is allowed on the total volume and peak flow. To meet this performance tolerance, base flow loading and/or the diurnal profile may be adjusted to match measured flow data.

Wet weather flow calibration is achieved by comparing peak flows, velocity and depth of metered data with a model's simulated results. Wet weather parameters, such as contributing runoff area, slope and width of drainage subcatchments, are adjusted until an acceptable correlation between simulation results and metered data is achieved for at least three recorded storm events. Calibration storm events have the following desirable characteristics:

- Antecedent conditions - preferably 5 dry days preceding the rainfall event.
- Total rainfall depth – 1-inch to 2-inch total depth. Larger storms (>2-inch) may produce manhole overflows within the model simulation results. This simulated flood volume loss will not be reflected in the measured flow meter data. Smaller storms (< 1-inch) do not adequately mimic the effects of a design storm.
- Season of event – Spring storms are preferable due to their tendency to be larger in volume and duration.

For wet weather calibration, a 20% margin of error is allowed.

Figure 2 illustrates how altering the runoff parameters generates a different model response that more closely resembles the actual meter data. The blue/green lines indicate meter data while the orange line indicates the results of the initial model simulation and the yellow line represents the results of the final model simulation.

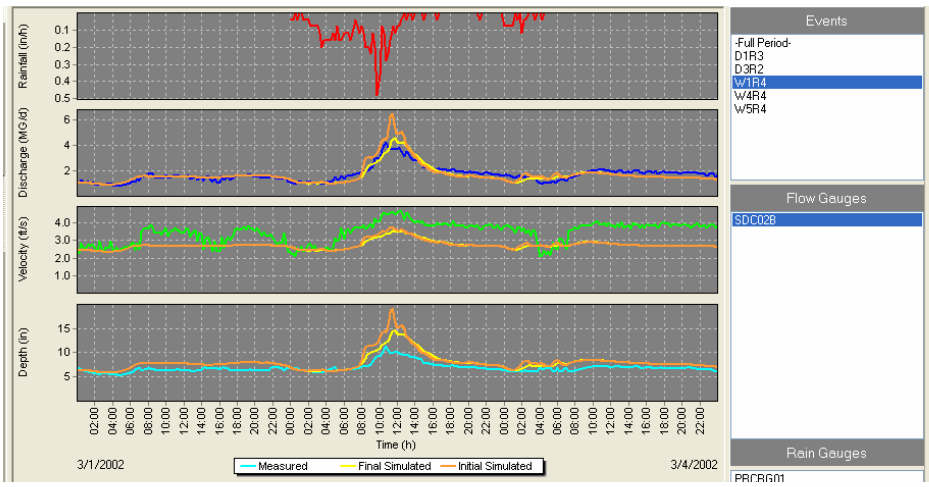

Figure 2 - Calibration Graph

Model Design Criteria

Design Storm

Although a design storm interval is specified for determining capacity in the system per the FACD, the storm itself is not defined. A 50 year analysis of historical rain data obtained from Atlanta's Hartsfield-Jackson International Airport revealed that there are two types of storms that occur in Atlanta; 'cloudburst' storms, which are short-duration, high-intensity, limited-area storms, and 'synoptic' storms, which are long-duration, large-area storms (Humphrey 2006). A 'synoptic', 2-year 24-hour storm was created for planning purposes. The majority of the 'cloudburst' storms in Atlanta occur over 1 to 3 hours. The cloudburst storm for a 2-year 3-hour event from Georgia Storm Management Manual (GSMM) has a peak intensity of 3.96 inch per hour and a total rainfall of 2.55 inches without DARF. The Hartsfield-Jackson Airport 56-year data is consistent with GSMM. As this storm generally occurs over a limited area, Depth Areal Reduction Factors (DARF) can be applied based on the size of the geographic area under analysis. By applying DARF, depth and peak of rainfall are reduced as the geographic area of the model grows, allowing for a realistic depiction of rainfall effects on the sewer system. Table 1 illustrates the effects of DARF on each drainage basin.

Basin	Area (sq mile)	Rainfall		DARF (%)	
		Total (inch)	Peak* (in/hr)	Total	Peak*
Long Island	3.10	2.49	3.77	97.6%	95.2%
Sugar Creek	4.64	2.44	3.60	95.7%	90.9%
Camp Creek	6.04	2.40	3.45	94.1%	87.1%
Sandy Creek	6.90	2.38	3.37	93.3%	85.1%
Intrenchment Creek	7.63	2.35	3.26	92.2%	82.3%
Nancy Creek	13.47	2.20	2.81	86.3%	71.0%
Proctor Creek	16.85	2.13	2.58	83.5%	65.2%
South River	18.64	2.09	2.47	82.0%	62.4%
Utoy Creek	22.34	2.02	2.28	79.2%	57.6%
Peachtree Creek	32.29	1.86	1.87	72.9%	47.2%
City Of Atlanta	**132**	**1.13**	**0.80**	**44.3%**	**20.2%**

*Peaks are based on 15-minutes time step.
(Center Loading, based on GSMM and 15 Minutes time step)
with DARF

Table 1 - Statistics of the 2-Year 3-Hour Cloud Burst Design Storm

The 'cloudburst' and 'synoptic' storms are realistic design storms which can accurately portray typical storms within the City. These storms are used for capacity analysis of the existing system and sizing future capacity relief projects.

Infiltration/Inflow (I/I)

Estimated amounts of I/I reduction have also been evaluated for design of capacity relief projects. Flow monitoring special studies have been performed on small sections of sewersheds in Camp Creek and Proctor Creek basins before and after completion of rehabilitation to quantify the amount of I/I reduction. While 20% I/I reduction is traditionally accepted, I/I reduction amounts upwards of 55% were observed in pipes alongside creeks that may be susceptible to large amounts of inflow. Based on the flow monitoring studies~~,~~ I/I reduction rates between 0 to 40%, with a typical value of 20%, are used with the knowledge that 100% reduction is not feasible as all rehab and pipe replacement work is performed primarily on the City side of the system.

Model Applications

To determine the capacity limitations of the existing system, a calibrated model is run under the cloudburst design storm. If while subjected to the design storm, the model results show surcharging, the system upstream is considered 'Capacity Limited'. Figure 3 illustrates the extent of capacity limitations in the existing system.

To evaluate the effectiveness of given alternatives to alleviate capacity limitations, the calibrated model with growth projections for the year 2050 are run under various scenarios for different design storm occurrence levels. For each scenario, any capacity limited pipes are upsized to accommodate the flow to 2/3 depth (<36 inches) and full depth for trunk lines (>36 inches) and storage is sized to reduce the peak flow reaching the treatment facilities. Each scenario is subjected to 40 years of historical rain data to quantify the amount and volume of overflows that would still occur if the projects were in place. This overflow data is combined with cost data for each project and plotted to determine the most cost-effective design storm and I/I reduction levels to build against.

As stipulated in the FACD, the peak flow reports must be submitted to show capacity limitations. Within this report conceptual relief projects that address limitations are provided. These reports define the extent of capacity issues in a basin and propose capacity relief projects along with an implementation schedule. Due to the potential stipulated penalties- as Paper One noted, the accuracy and timeliness of these reports was crucial.

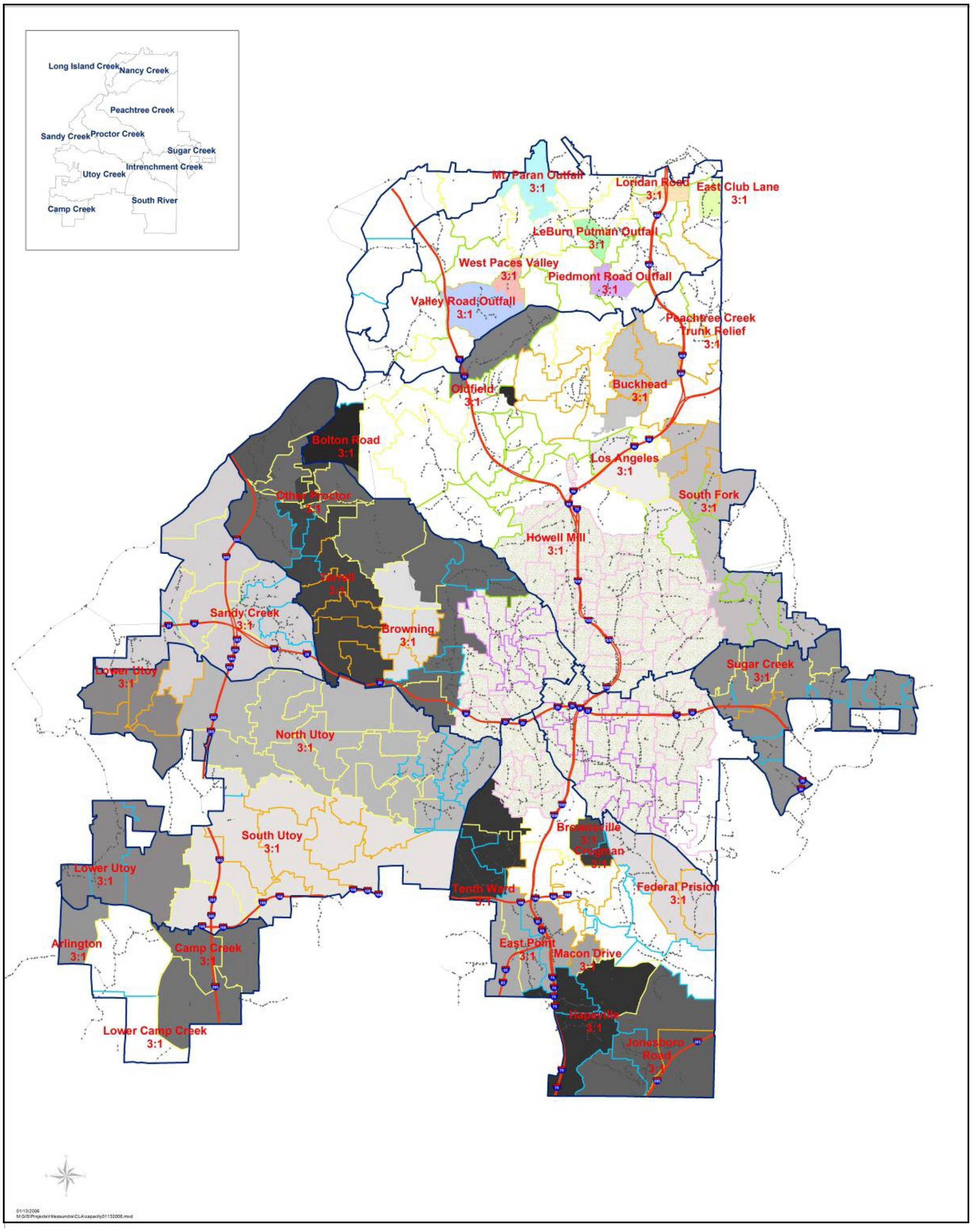

Figure 3 – City of Atlanta Capacity Limited Areas

In addition to the larger capacity relief projects, the model is also used for the development of pipe replacement proposals for upsizing smaller diameter pipes. It is also used to respond to Requests For Information (RFIs). These RFIs come from several sources including developers requesting solutions to capacity issues and the City law department needing assistance in determining the cause of overflows.

Status

In 2005, the City re-evaluated its modeling software usage. A decision to convert all models from XP-SWMM to Infoworks CS (formerly Hydroworks) was reached for a variety of reasons; stability and reduced run times for very large modeling simulations, GIS compatibility, enhanced model output reporting and general ease of use. The City implemented a plan to convert all basin models. This minimized the amount of time that a basin model was in transition and resulted in a seamless conversion. Amidst this software change and a review of the modeling design criteria, the task is on schedule with currently 954 of the City's 1900 miles of sewer incorporated into the model.

In accordance with FACD requirements, Peak Flow Evaluation Reports for all ten separated sewer basins have been submitted. Four of the six Sewer Group Peak Flow Evaluation Reports have also been submitted, which estimate the magnitude of I/I for each sewer group by defining the hydraulic characteristics of each sewershed within each sewer group as well as separating I/I from base flow.

In addition, the modeling task supports numerous deliverables in the Clean Water Atlanta initiative- as Paper One noted. For example, where the FACD-mandated Peak Flow Evaluation Reports lay the backbone of capital improvement projects, the SSO Remedial Measures Plan more thoroughly refines these projects. Of the ten reports to be submitted, one has been accepted by the City with three more awaiting finalization. The remaining seven reports shall be submitted to the City by April 2008.

Once rehabilitation and replacement work is completed, the model will be updated from sewershed 'As-built' map drawings.

Conclusions

Further improvements are underway to increase the accuracy and efficiency of the model building, calibration and capacity relief project evaluation process:

- Volume balance analysis of monthly flow monitor submissions to check the quality of data and, if needed, recommend corrective actions to flow meter contractor.
- Review SSES defect information to provide a range of I/I reduction rather than applying a constant 20% I/I reduction value across the entire sewer basin.
- If capacity of existing 8-inch pipe appears insufficient, verify within the model that there are more than 300 houses upstream, or an equivalent dry

weather flow from mixed land use properties, of the proposed upsized pipe segment.
- Due to the homogeneity of demographic information across census tracts, use GIS land usage information to help apply the most accurate growth projections when modeling sewershed-sized service areas.
- All models would benefit from expanding the rain gauge and flow meter network.
- The City is currently installing an automated water meter network that would provide residential consumption and trade flow information. This data, would allow us to have a verified model rather than calibrating our model network to only flow monitor data;
- Improve consistency and volume balance of flow readings between the contractor-managed flow monitor network and the city-managed interjurisdictional monitors.

In the upcoming years, the model will continue to be a tremendous asset in achieving FACD deadlines from being a resource for the design of capacity relief projects to identifying the amount of flows that can be attributed to the six inter-jurisdictional municipalities that feed into the system.

References

Humphrey, John J. (March 2006). *Atlanta Design Storm Processor.* Report prepared for the Clean Water Atlanta Program.

United States District Court Northern District of Georgia, Atlanta Division, Civil Action File No. 1:98-CV-1956-TWT (1999)

Fast-Track Pipeline Rehabilitation Using a Carbon Fiber Reinforced Polymer (CFRP) Strengthening System

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Abstract

Internal pipeline inspections can often lead to observations of distress and emergency action must be taken to get the pipeline back into service. In many cases, the pipe does not wait for an inspection to demonstrate signs of distress. The San Diego County Water Authority has first hand knowledge of these situations and has needed to act quickly in several cases to get the pipeline back into service. On several occasions, the fastest and most viable technique was Carbon Fiber Reinforced Polymer (CFRP) strengthening. When a pipeline needs to be strengthened quickly, few alternatives have proved as fast as CFRP lining. The design and installation of these materials must be done by specialized engineers and contractors using systems that have been thoroughly tested. Many projects have been completed that presented unique challenges. This paper will review the project details, specification requirements and lessons learned from the completed installations.

A Brief History of Previous Projects

The San Diego County Water Authority schedules internal pipeline inspections to gather condition assessment data and to make planned repairs to sections of the pipeline that are nearing the end of their service life. During these planned inspections/repairs, there are times when unexpected problems are found and immediate repairs are necessary. Since the pipelines can only be shut down for a maximum of 10 days, these repairs are generally done around the clock and must be completed within 48 to 96 hours. This proves to be challenging when the repair includes large diameter PCCP (66 to 96 inch).

Up until December 2003, the Water Authority repaired these distressed sections with steel pipe. This involved excavating a large hole, removing the damaged section (s), and installing a welded steel pipe section. This type of repair impacted the surrounding environment, public, traffic, and generally took longer than 96 hours. On December 10, 2003 while performing an internal inspection, a crack was discovered on the inside of an 84-inch diameter section of PCCP. The crack started at the upstream joint for a length of approximately 2-feet. The mortar lining was removed and a 1.5-foot crack in the steel can was located and groundwater was coming back into the pipe (see Figure 1 below).

Figure 1 - Urgent Repair Pipe Mark 543

Based on the location of the pipe (adjacent to homes), the close proximity of a manhole, and the relatively low-pressure (60 psi), a carbon fiber application was selected for the repair method. The contractor was notified on Thursday and the crew and equipment began to mobilize that night. Welding and repair work were completed Friday. Surface preparation of the section and the beginning of the wrapping took place on Saturday. Wrapping was completed on Sunday and all equipment and scaffolding inside the pipe was removed late Sunday night. Dehumidifiers were used to accelerate the cure for an additional twelve hours prior to turning the pipe back over to the Water Authority. The team of two crews worked around the clock to complete the repair within four days. This was the first time the Water Authority utilized this method for a repair.

In May 2006, the Water Authority had a failure on a section of 66 inch PCCP that caused an unscheduled shutdown of the untreated water pipelines. The information gathered from this failure led to a re-evaluation of electromagnetic data and two sections of 96-inch PCCP located in the northern section of San Diego were selected for further investigations. To perform these investigations, the Water Authority scheduled a six day untreated water shutdown starting on Sunday, June 11, 2006, to visually confirm the corrosion damage. On Monday, June 12, a visual inspection of the exterior and interior of the pipe was performed. A total of 33 wires were broken and 95 wires were corroded (see Figure 2). A structural analysis was performed and showed that the excavated pipe as well as one additional pipe needed to be repaired.

Figure 2 – Post Excavation and Removal of Mortar Lining

Figure 3 – Damaged wires and corroded area

Due to the urgent nature of the work, the favorable site and pipeline access location, and the tight schedule, the repair technique of choice was carbon fiber lining. On Monday June 12, at 2:00 p.m., an urgent repair contract was awarded to repair two 96-inch PCCP sections.

The contractor mobilized and began work at 6:00 p.m. on Monday June 12, 2006. The work included the interior cleaning of the pipeline, installation of the carbon fiber, curing of the fiber, and installation of a potable water compatible coating. (See Figures 4 through 8)

Figure 4 – Interior Cleaning of the Pipe Surface

Figure 5 –Carbon Fiber Material Application

Figure 6 – Carbon Fiber Equipment

Figure 7 – Application of the Potable Water Coating

All work was completed on Thursday, June 15, 2006, and the pipeline was filled and placed back into service mid afternoon on Friday, June 16, 2006.

Figure 8 – Finished Carbon Fiber Mark 240

Design, Detailing and Project Logistics

The inspection, data interpretation and the development of the design criteria for the identified pipe sections are the responsibility of SDCWA. The design and detailing of these proprietary CFRP systems is supplied by the specialty contractors and is then reviewed by SDCWA. There is no current AWWA standard for the design of these systems, so each system must be validated by independent structural and durability testing to ensure quality. The specialty contractors and their consultants must take on the liability associated with the design of the systems.

The design parameters and required safety factors must be clearly defined for each individual project. These parameters will vary from project to project depending on the internal pressure, the depth of the pipe, the location of the pipe and the target service life of the repair. Projects such as the one in Scripps Ranch did not require as much conservatism due to the fact that the repair was only needed to provide one more year of service prior to the section being slip lined with steel as part of a planned shutdown. The more layers required for a particular project, the more time it takes for the installation.

It seems that all projects come with some unforeseen events and/or field complications. The Scripps Ranch project proved to be more challenging then expected with respect to the repairs that were required prior to the wrapping. Exposing the fractured steel cylinder revealed the need to weld the damaged section. The new steel plate was rolled at the shop in order to match the curvature of the existing cylinder. The plate delivered to the site did not fit as planned and field modifications were required to successfully stop the ground water from penetrating the cylinder. The welding process lasted several hours more than the crew expected (see Figure 9) but the project stayed on schedule.

Figure 9 – Welding Repair of Fractured Steel Cylinder

The project access area was within only a few feet of residential housing. The Scripps Ranch residents were not pleased with the 24-hour operation and the associated lights and noises. However, the overall impact on the public was substantially minimized by the speed of the operation and the fact that no excavation work was required.

The Twin Oaks Valley project was the first time we had attempted to chip out the end joints and create a watertight barrier between the steel cylinder and the CFRP liner. The chipping process was much more difficult than expected and took more than twice as long as the crew had expected. The new detail caused some initial confusion but the installation was successful (see Figure 10).

Figure 10 – Sealing of the Liner to the Steel Cylinder

Previous installations have shown no sign that there is an issue with the internal water pressure on the steel cylinder due to the presence of the CFRP liner. There is an extra day or more that is required to detail the liner such that it has a watertight seal to the steel liner at both ends of each repaired pipe section. Further testing and/or observation of existing installations may show that this additional work is not required but SDCWA considers it to be worth the additional time and cost to have a more conservative detail.

Possibly the most important component of a well-coordinated and fast installation is the experience and skill level of the specialty contractor and their team of engineers and material providers. The contractor must be able to maintain a safe working environment on 24-hour operations, while coordinating the installation and the rapid procurement of the required materials. The ability of the contractor to work seamlessly with both his material supplier and his engineers is critical to a fast-track repair. The materials must be in stock and ready to ship at a moments notice. In addition, the labor force and engineering team must be able to coordinate the design details and logistics quickly.

Conclusions

There are a limited number of solutions to a critically damaged pipeline. Many factors need to be considered when choosing the best method of repair but the two most important factors are the cost of the repair compared to the time that the pipeline needs to be out of service.

The CFRP lining solution has proven to be useful in certain types of scenarios. The Water Authority has used this repair method on seven sections of PCCP ranging in diameter from 84 to 96 inches and pressures from 60 to 200 psi. Because of the success of these repairs, the Water Authority is investigating the use of CFRP lining on longer lengths of pipeline as competition to the steel slip lining that has been used since 1982.

New detailing techniques and construction methods combined with improved materials, could lead to a faster and more efficient process and subsequently more CFRP lining projects in the future.

Trenchless Rehabilitation of Large Brick Conduits in Boston

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Introduction

The Boston Water and Sewer Commission is a public agency created in 1977 by the Commonwealth of Massachusetts. Its enabling legislation gave them ownership of the water, sewer and drainage systems within the City of Boston. Since its inception the Commission set several goals for improvements to those systems through a clearly defined annual Capital Improvement Program. Since 1978 the water, sewer and drainage systems in the city have seen substantial improvements in the areas of water quality and reductions of Combined Sewer Overflows to Boston Harbor and its tributary rivers. Hundreds of miles of water and sewer pipe have been replaced or rehabilitated. More than eighty miles of new drains have been installed for the purpose of sewer separation. New larger sewer interceptors have been constructed along the waterfront and many sewer and drain interceptors have been cleaned and inspected. The results of the inspections have revealed that most of the systems large pipes are in good condition with the exception of some of the old brick sewers and drains that have exhibited various minor structural problems or infiltration due to degradation of mortar. Rehabilitation of odd shaped brick conduits presents a difficult challenge to provide adequate and long term repair to the conduit at the lowest cost and amount of disruption to the surrounding neighborhoods. Common rehabilitation methods are often not applicable to large or odd shapes and pipe replacement may not be a viable option. This paper presents the planning, design and construction of two projects in Boston involving the rehabilitation of large brick conduits.

The Old Stony Brook Conduit

The Old Stony Brook Conduit was constructed in 1904 in the Roxbury section of Boston. It varies in size as it meanders along the original stream bed of the same name. The conduit was constructed as a combined sewer but, after upstream sewer separation, now functions as a storm drain The conduit was cleaned of all sediments in 1999 under a publicly bid contract which also included an inspection of the pipe after completion of the cleaning. Much of the inspection was completed using a hand held camera while walking through the conduit. The inspection tape was reviewed by Commission staff and it was found that the conduit was in very good condition with the exception of approximately 1,500 feet of the conduit. At that location the pipe measured 132"x 90" and was constructed of three courses of overlapping bricks. The conduit was constructed as an arch with a spread base and a slightly concave floor which was typical for that period. Figure 1 shows a cross section of the conduit.

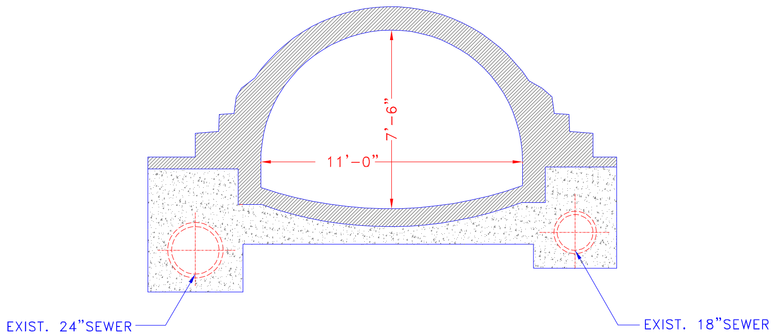

Figure1. Old Stony Brook Conduit

As shown in the figure the local sewers were constructed within the foundation of the drain and covered in poured concrete.

The original inspection videos showed a large crack along the crown of the conduit which would indicate a loss of lateral support along the sides of the arch. This condition is typical in brick conduit construction. Inspection of miles of brick conduits in Boston shows this same condition in many pipes. The cracks however are generally small and generally haven't caused significant loss of strength to the arch and thus to the conduit as a whole. The construction of brick sewers is typified by a high quality of workmanship in construction of the conduit. Later excavations around the sides of the conduits or other disturbances to the backfill adjacent to the conduits might lead to uneven lateral support to the sides of the arch which cause minor cracks. These can often be repaired by grouting the cracks, filling the cracks with hydraulic cement or installing a liner within the conduit.

A closer inspection of the cracked section of the Old Stony Brook Conduit revealed that the crack was severe and in some locations extended through all three layers of the bricks and opened up to the soil. A factor which caused some alarm was the shallow depth of the top of the conduit from the street surface which ranged from 18-inches to 3-feet along the length of the damaged sections of conduit. Figure 2. illustrates the severity of the cracks and the effects of the damage. Most notable is the downward deflection of the crown of the pipe. During the design of the pipe rehabilitation method a crack developed in the street above the crown of the conduit. A second inspection revealed that the crack in the crown of the conduit had become more severe which prompted the Commission to close off a portion of the street with Jersey Barriers. It is evident that this condition required a structural repair or reconstruction of the conduit.

Figure 2 showing a separation at the crown of the conduit

<u>Alternative Methods of Rehabilitation</u>

The initial phase of any trenchless project requires a comparison of all alternative methods of rehabilitation including the possibility of complete replacement. Replacement of this size conduit requires an excavation which would encompass most of the street. The conduit lies beneath a residential street with moderate traffic which can be rerouted but the main concern was the length of time required to install such a large replacement pipe as well as new sewers and water mains. This method was put aside until all other alternatives were reviewed. Methods considered were:

Cured-in-place liner
Epoxy coating with pressure grout
Spiral wound PVC
Localized grouting and sealing with concrete
Shotcrete
Sliplining

Prior to the evaluation of alternatives it was necessary to determine the allowable loss of cross sectional area of the conduit and its affect on the hydraulic capacity of the conduit to carry storm flows for the required size storm for this area of the city. The conduit drains an area of approximately 300 acres and during a 10 year storm event requires a capacity of 300 cubic feet per second. Using this information it was determined that the internal diameter of the conduit could be reduced by 8-inches. This information set a parameter for review of alternatives as well as the final design.

Methods such as epoxy coating and grout sealing were discounted as not capable of providing structural stability to the conduit. They are effective means of rehabilitating pipe with non-structural deficiencies which was not the case in this situation. Spiral wound PVC was, at that time not a proven method for non-circular pipe in the United States and was thus discounted. Sliplining was considered a serious alternative but the loss in cross sectional area was greater than allowable. Cured-in-place liners are an excellent method of rehabilitation and have been successfully installed on large diameter conduits, many in Boston. There were however risks involved in trying to manufacture and install such an odd shaped liner in a conduit of this size. Also, the existing manholes are not large enough to insert a large liner, thus requiring excavation to open the top of the conduit.

Shotcrete is a concrete mix that is spray applied through a pump and nozzle system directly onto the brick pipe. It is usually reinforced with wire mesh attached to the inside of the conduit but can also be utilized with a steel reinforcement system designed for complete structural support. This is, in essence, the construction of an independent reinforced concrete pipe within an existing pipe with the flexibility to construct the new pipe to conform to the shape of the host pipe. This is not a new technology and has been used in similar projects in Boston since 1982. It is a viable method of structural rehabilitation for large conduits where flow can be diverted to allow man entry. In this case the conduit carries a small amount of dry weather flow from an upstream brook. That small flow could be pumped around the area of construction. The question remaining was could required cross sectional area be maintained as part of the final design.

The Commission hired the firm of Metcalf & Eddy of Wakefield, MA who with its sub consultant, Corrosion Probe, Inc. of Scituate, MA, provided a design for the reinforcement and cross section of an 8-inch thick concrete liner which would provide for both our structural and hydraulic criteria. With all criteria met a final cost analysis was performed comparing the Shotcrete solution with partial and complete reconstruction. It was determined that Shotcrete was the less costly and time consuming alternative and thus the decision was made to proceed with final design.

The final design incorporated the structural details provided by the consultant as well as detailed specifications for the materials and application of Shotcrete for this specific project. Figure 3 shows the detail of the cross section as designed.

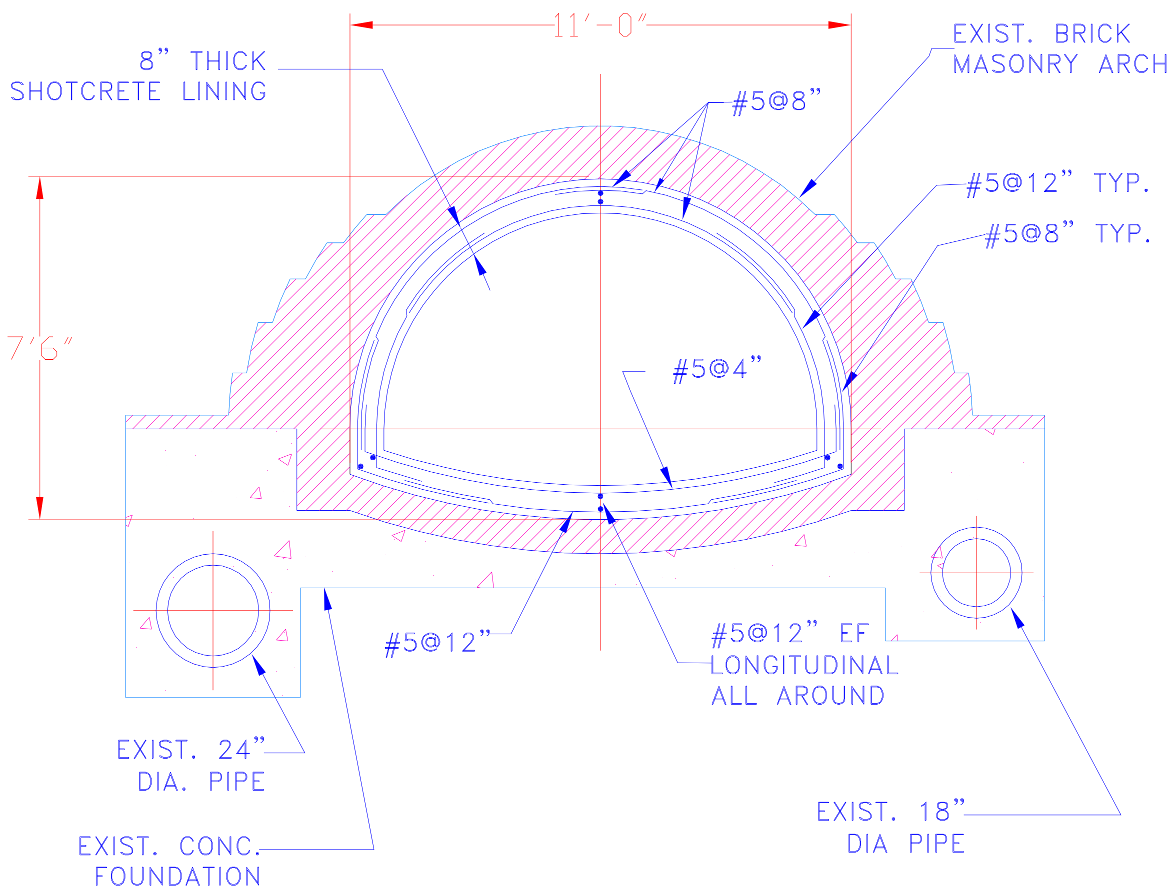

Figure 3

The need for immediate repair of the conduit to avoid possible street failure precluded the prequalification of contractors. Instead, criteria for acceptance as a qualified contractor were incorporated into the bid package and only those contractors who provided the required qualifications packages were allowed to submit a bid. The low bidder for this project was Barletta Heavy Equipment of Boston with a price of approximately $1.3 million or approximately $840 per linear foot. The estimated cost for replacement was estimated at $2,000 per linear foot. The low bid was accepted and work commenced shortly after the award of the contract.

The specifications for this work included independent testing of the concrete mixture and the taking of core samples throughout the length of the conduit to ensure that the product met the required standards. The specifications also detailed the amount of admixtures and aggregate for the concrete as well as the application process and the maximum thickness of each layer applied.

After review of the project schedule and the shop drawings the project was allowed to commence. In addition to an onsite inspector the design consultant provided inspection of each major phase of the work including a final inspection of the steel reinforcement before any application of concrete. They also oversaw the core sampling and the field sampling of the concrete mixture. The entire installation including the quality control measures was completed in 65 working days. Figure 4 shows the reinforcing steel installed prior to application of the shotcrete.

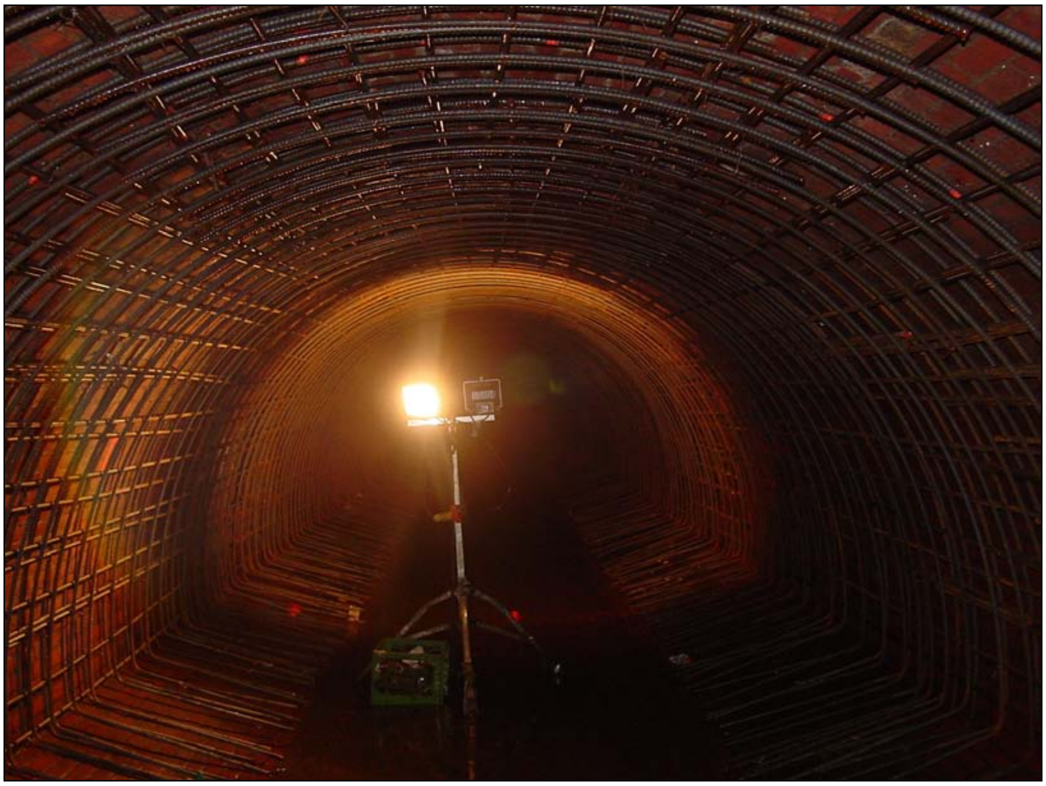

Figure 4 showing the steel reinforcing

The application process involved several application layers of shotcrete. Quality control was maintained to ensure proper placement and installation of the reinforcement. Each application of shotcrete was measured for proper thickness and the concrete mix was inspected for adherence to the specifications. After the final application the shotcrete is troweled leaving solid smooth surface. Figure 5 shows the final stages of the application process. At this time the bottom of the conduit was finished and the brook flow was allowed to continue through the conduit unimpeded.

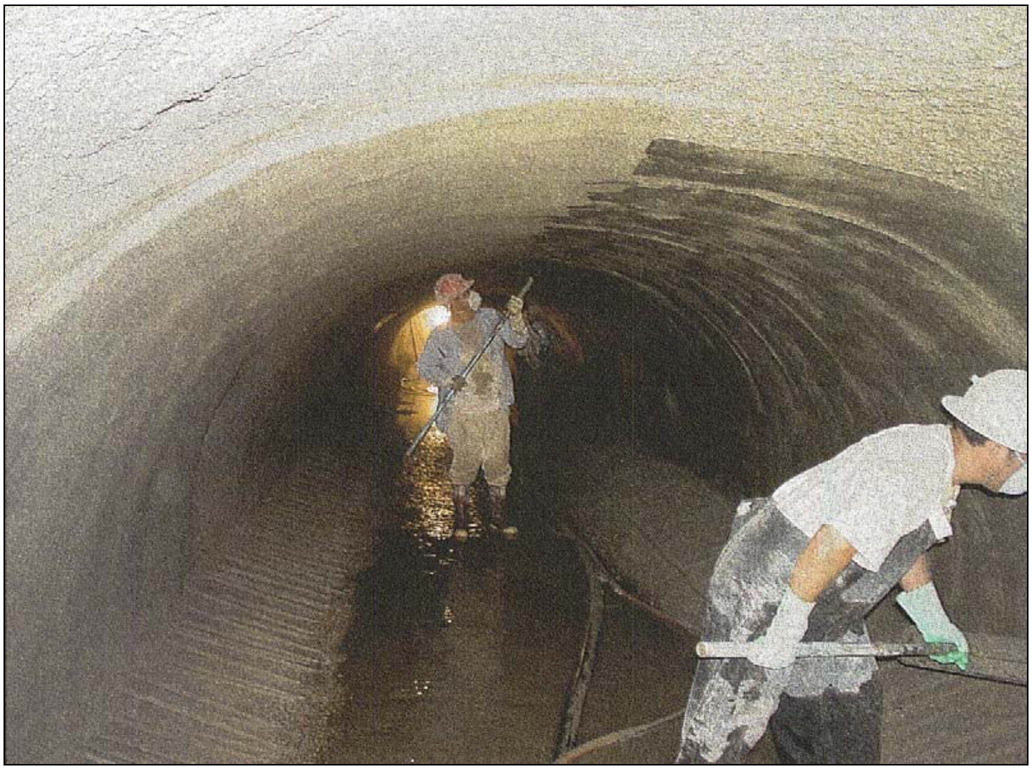

Figure 5

West Side Interceptor

The West Side Interceptor is a 57" x 66" inverted egg shaped brick sewer constructed in 1878 that carries sewerage and wet weather flows. The pipe was cleaned and inspected and although found to be structurally sound there were locations of active although minor infiltration requiring attention. The length of conduit requiring rehabilitation was approximately 4,150 linear feet. The sewer is located in a historic district in the Back Bay/Lower Beacon Hill Area of Boston. It is a residential area with a great deal of pedestrian traffic which increases in the spring and summer with the influx of tourists. Although the pipe was found to be in good condition with minor infiltration it was decided that as a vital part of the Boston Sewer system the pipe should be rehabilitated to ensure its long term integrity and to prevent any further degradation which would allow an increase in ground water infiltration. Figure 6 shows the cross section of the conduit.

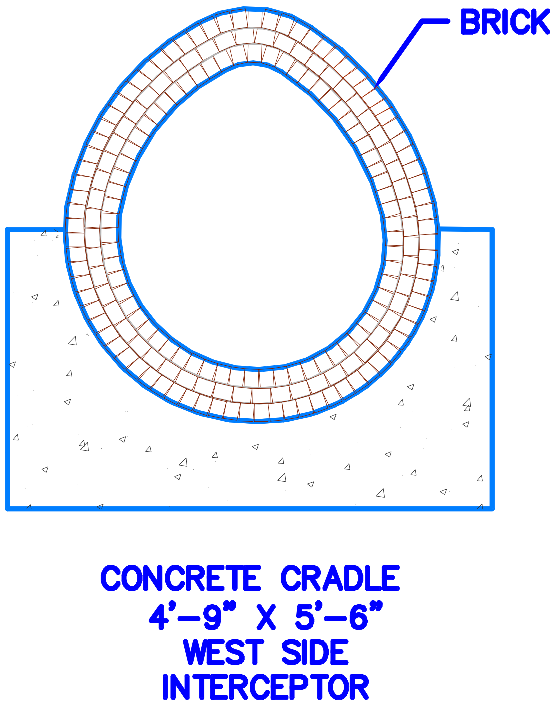

Figure 6

As with all large projects an analysis of alternatives was undertaken to determine the method of rehabilitation that would be most effective while minimizing disruption to the area. Due to the relatively good condition of the pipe and its location it was deemed unnecessary to consider replacement as an option. There were several trenchless methods to be considered including:

Cured-in-place lining
Epoxy coating
Cementitious coating with epoxy
Slip lining
Spiral wound lining

The first consideration was to decide whether to provide a system to only eliminate the infiltration or to provide for a full rehabilitation. Due to the importance of the interceptor it was deemed prudent to fully rehabilitate the pipe without sacrificing necessary flow capacity. During recent construction of the new underground Central Artery the upstream portion of this interceptor was rerouted eliminating 5 acres of residential area from the system. Upstream sewer separation eliminated an additional 30 acres or 25 million gallons per day of peak wet weather flow. In order to quantify the dry weather flow two sewer meters were placed in the conduit and measured an average of 4 million gallons per day (MGD). The conduit has a capacity to carry 32 MGD. This pipe is a combined sewer with overflow locations that flow into a larger conduit (The Boston Marginal Conduit) owned by the regional water and sewer authority, The Massachusetts Water Resources Authority. The Marginal Conduit flows to a treatment facility prior to discharge into the Charles River. With the loss of a considerable amount of upstream dry and wet weather flows it was determined that there was available capacity in the sewer to allow for a decrease in cross sectional area. This allowed for greater flexibility in evaluating alternatives.

Alternatives involving the placement of PVC linings and panels were considered due to the ability to place these without bypass of all flow. There was however a need to temporarily divert some of the flow in order to install these systems. The only method that would not require flow diversion is sliplining. This method was eliminated because of the need for large excavations at insertion points. The intent was to rehabilitate the pipe with as little excavation as possible. The streets are clogged with utilities making excavation difficult. Also, concerns with noise and traffic control helped in the decision to eliminate any method that would require excavation. It was at this point necessary to determine the best method to bypass flows during construction. The normal method of bypass pumping was achievable but would require large pipes to be installed in shallow excavations to carry the flow and a fairly large pumping system that would need to run on a continuous basis. After analysis of the available network of sewers and drains in the area it was decided to approach the Massachusetts Water Resources Authority (MWRA), to request temporary use of their nearby Boston Marginal Conduit, which is connected to the West Side Interceptor at several locations The conduit carries a small base flow leaving a great amount of available capacity during dry weather. With the concurrence of the MWRA the flow in the West Side Interceptor was diverted in the Marginal Conduit where a temporary low dam forced the flow in the opposite direction. With the flow reversed, it returned to the Boston system downstream of the work zone thus leaving the interceptor completely dry with pumping only occurring within a manhole adjacent to the Marginal Conduit.

With the matter of flow diversion resolved the alternative methods could be reviewed in a different light. One of the methods used to rehabilitate large diameter pipe if the installation of spiral wound PVC. This can be used with non circular pipe and constitutes a structural repair. This is a trenchless method that has not been used in Boston but is under serious consideration. This method was analyzed but the installation procedure for a non circular conduit would leave a large annular space between the new liner and the host pipe that would need to be filled. This might reduce the cross section to an unacceptable level. This method was set aside as a possibility.

The Boston Water and Sewer Commission has experienced a mixture of success and failure with the use of epoxy coatings. In order to evaluate the effectiveness of these materials we have retained contractors who specialize in the use of these materials to apply them in certain locations where they can be monitored. Our general conclusion is that the application of solid epoxies directly onto manhole walls is effective but this we have only found one contractor who specializes in the application of a pure solid epoxy. Epoxy coatings have been effective applied over a newly applied cementitious base for manholes but when this method was used for a brick sewer in Boston under a test trial situation it failed after a short period of time. While the sole use of a solid epoxy was considered we were not convinced of its long term durability. With considerable expense being applied to flow diversion it was decided that it was not worth the risk to try an unproven product at this location. The final decision was to prepare plans and specifications to rehabilitate the pipe using a cured-in-place liner. This method has been used successfully in Boston since 1982 on both circular and odd shaped pipes although not on a pipe of this large size.

The plans and specification detailed the work plan for both the flow diversion system and the specifications for the lining. The bid documents included detailed requirements for acceptable contractor experience and references which must be approved prior to acceptance of bids. Although the liner was to be installed within an intercepting sewer there were a small number of building connections that would have to be bypassed during the installation and curing process. The curing process for large diameter pipes is longer that for small diameter pipes and thus the building connections may need to be temporarily pumped into an alternate system. These services were covered in the bid documents and paid for as a lump sum item. Bids were received and the contract awarded to Spiniello Construction Company of New Jersey, the low bidder for a price of $2,885,960.00 or 695.00 per linear foot. The price included all work including flow diversion and traffic management.

The construction of this project was not without difficulty as the installation and curing of large liners is a long continuous process requiring the contractor to be on site day and night. Machinery that was required to run at night resulted in complaints from abutters but the time spent at a given location was short which mitigated the problem. Anther issue arose when complaints were received of strong odors in a building containing a restaurant. Odors emanating from the curing process had entered the basement of a building with inadequate plumbing that did not trap the gas. This issue was dealt with by the contractor but must become a consideration in future contracts during the design phase. Otherwise the project was successfully completed without major difficulty. Figure 7 shows the interior of the newly lined conduit.

Figure 7

Conclusion

The rehabilitation of brick conduits must take into account the condition of the pipe with respect to its structural integrity and hydraulic capacity. Some projects are designed only as a measure to preserve the integrity of a pipe in good condition or to eliminate infiltration. There are a wide variety of rehabilitation methods available for these goals some of which can be used simultaneously. For brick conduits, especially those with non circular shapes, the alternatives are few and require sound engineering judgment to determine the correct methods.

Pipelines, Trains and Automobiles: Rehabilitation of an 18" Sewer with No Excavation - Howard Street Sewer Project, Framingham, MA

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Abstract

As part of a sewer replacement project, the Town of Framingham, Massachusetts determined the most effective construction method to repair a failing 18-inch sewer located directly underneath a heavily traveled Massachusetts Bay Transit Authority (MBTA) and CSX Corporation (CSX) railroad, and located in a downtown area with high vehicle and pedestrian traffic. The rehabilitation of this sewer was made without any excavation, by slipping a new 12-inch replacement pipe inside the old failing 18-inch sewer, essentially creating a new pipe inside the old failing pipe.

Project Background

The Howard Street Sewer Project in Framingham, Massachusetts was a critical first step for the Town of Framingham to redevelop its downtown area, beginning its transformation from an area of urban blight to a vibrant downtown filled with residential and commercial development. The Howard Street Sewer Project provided the necessary sewer infrastructure improvements and associated flow capacity to accommodate the downtown redevelopment plans.

The Howard Street Sewer project consisted of the design and construction of 2,500 linear feet of new 18-inch PVC sewer, replacing aged and deteriorating 15-inch vitrified clay pipe in downtown Framingham. This Project also included the repair of 200-feet of existing 18-inch vitrified clay pipe. This 200 foot section of pipe was failing, with a large rock protruding through it, and located underneath Route 135 (4 traffic lanes) and the MBTA/CSX main line railroad. A repair was required to prevent further damage to the pipe, and to prevent a potential sinkhole from being created under the railroad tracks. It is noted that this railroad supports the heaviest volume of passenger and freight traffic through the State of Massachusetts. As such, the rehabilitation of this section of existing sewer presented a significant challenge to the Town; the need to expeditiously identify and implement a repair that would not jeopardize or disrupt the operations of the railroad, and ideally not interfere with the vehicular and pedestrian traffic associated with Route 135 and the surrounding downtown area.

This report details how the Town of Framingham faced this challenge and successfully rehabilitated this critical section of sewer pipe. It presents the ***approach*** and ***alternative analysis*** used to determine the appropriate rehabilitation method for this pipeline; and details the successful ***construction method*** used to complete the pipeline rehabilitation.

Approach for Determining Rehabilitation Method

Determining the most appropriate rehabilitation method for repairing the section of sewer located underneath Route 135 and the MBTA/CSX main line railroad was difficult due to the following factors:

- Lack of Existing Pipeline Condition Information
- Challenging Design Conditions

The following approach was utilized to address of each of these factors and ultimately select and implement a rehabilitation method.

Lack of Existing Pipeline Condition Information

Obtaining existing condition information about the pipeline was difficult because the pipe was normally flowing in a full or surcharged condition and could not be readily bypassed. As such, initial video inspection could only be done with significant volumes of wastewater flowing through the pipeline, making it difficult to document the exact condition of the pipe. However, these initial video inspections did reveal some obvious infiltration locations, and also indicated that a rock was protruding into the pipeline. Unfortunately, the video inspections were not sufficient to provide design criteria for selecting a rehabilitation technique. Given this limited information, only the following conclusions could be made:

- Further existing condition information would be required to determine if the pipeline could be repaired or if permanent abandonment was necessary
- A sewer by-pass would be required to remove the flow from this pipeline and allow for the collection of further existing condition information

Given this, the approach for determining a rehabilitation method for repairing this section of pipe required that a permanent by-pass sewer be constructed as part of the overall Howard Street Sewer Project. The by-pass sewer would direct flow away from the pipeline under the railroad tracks and into the Town's nearby transmission sewer. This new configuration would make the pipeline under the railroad track inactive, allowing for further video inspection to be conducted. The further video inspection would provide the information necessary to determine if the pipeline under the tracks could be repaired or if permanent abandonment was necessary. The Town recognized the value of maintaining a pipeline under the railroad tracks, and therefore, did not want to permanently abandon this sewer if at all possible.

As such, also included in the overall Howard Street Sewer Project was the performance of a further video inspection of the pipeline under the railroad tracks, once flows were bypassed, in order to obtain more detailed information on the condition of the pipeline.

Once completed, this further video inspection did provide greater detail with respect to the condition of the pipeline. Unfortunately, the greater detail of the video inspection indicated that the pipeline was in worse condition than originally thought. A clear picture of the protruding rock was obtained with this inspection, which is shown in Figure 1 – Rock Protruding into Existing Pipeline. With this new information about the condition of the pipeline and the extent of the protruding rock, a thorough alternatives analysis could be conducted to determine if the pipe could be rehabilitated and if so, identify the most appropriate rehabilitation method.

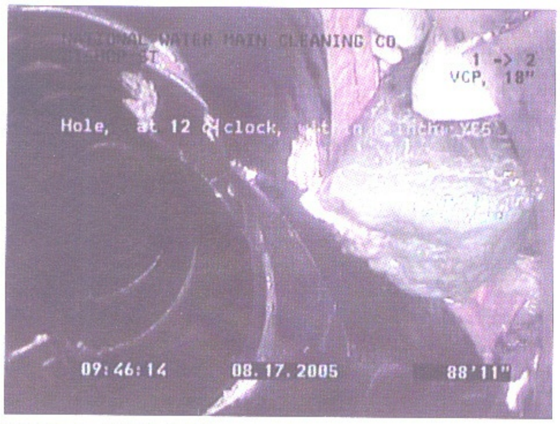

IMAGE: 9a, TAPE #: TV115-081705-1, 00:14:11
88.11FT. Hole, at 12 o´clock, within 8 inch: YES

Figure 1: Rock Protruding into Existing Pipeline

<u>Challenging Design Conditions</u>

The alternatives analysis had to account for the challenging design conditions associated with the rehabilitation of this section of sewer pipe. As previously indicated, the Town needed to expeditiously identify and implement a repair that would not jeopardize or disrupt the operations of the railroad, and ideally not interfere with the vehicular and pedestrian traffic associated with Route 135 and the surrounding downtown area.

Therefore, it became evident that the pipeline <u>must</u> be rehabilitated utilizing a trenchless construction method that would not affect the rail, vehicular or pedestrian traffic; and not compromise the integrity of the train tracks or roadway.

Fortunately, there were two factors that allowed for the consideration of several trenchless technology options. First, the existing pipeline had no sewer services, which would have required excavation or coring, to connect to a new pipe. Second, because a permanent by-pass sewer was constructed, the existing pipeline was being rehabilitated to serve only as a "spare" or overflow pipe. Therefore, the diameter of the pipeline could be reduced. It is noted that most repairs aim to maintain or increase the existing diameter of the pipeline.

Alternatives Analysis

Several trenchless construction alternatives were considered and evaluated to determine the most appropriate rehabilitation method for repairing this section of sewer.

The first alternative considered was rehabilitating the pipeline by installing cured-in-place pipe (CIPP) lining. However, it was not certain that the protruding rock could be displaced out of the pipe, and if not, the rock would create a deformity in the CIPP lining that would compromise its structural integrity. As such, the CIPP product manufacturer could not guarantee the structural integrity of the liner. Therefore, this alternative was eliminated due to the uncertainty of the installation and the inability to guarantee the integrity of the repair.

The next alternative considered was to utilize a hydraulic jack to displace the protruding rock out of the pipe and back into the surrounding fill; followed by the installation of a CIPP liner to rehabilitate the pipeline. The use of a hydraulic jack for such purposes is more common on larger sized pipes, where a person can fit inside the pipe and manually operate the hydraulic jack equipment. Given an inside diameter of 18-inches, a remotely controlled jack would be required to perform this task. After several discussions with trenchless contractors, this alternative was eliminated because it would have been too difficult and likely expensive to obtain a hydraulic jack that could be used remotely inside this pipeline. Furthermore, it was uncertain if the force of the hydraulic jack would damage the existing pipeline, further deteriorating the condition of the sewer.

Repairing the pipeline with an open trench excavation at the point of the rock protrusion was also considered. The rock protrusion could have been eliminated with the open trench point repair, followed by the installation of a CIPP liner to rehabilitate the entire pipeline. This alternative required excavating about 12-feet deep at the point where the rock protruded into the pipeline, which was unfortunately located directly beneath the railroad tracks. As such, this alternative would have resulted in extensive disruption to the CSX / MBTA railroad operations and would most likely not be allowed by these entities. Therefore, this alternative was eliminated.

Another alternative considered was pipe bursting the existing pipeline, and replacing the pipeline with a new high density polyethylene (HDPE) pipe. This alternative would be feasible because there were no sewer services located on the pipeline, and this method would have resulted in a new pipe the same capacity or greater than the existing pipe. Also, the protruding rock would likely be pushed back or broken up during the pipe bursting operations. Pipe bursting requires a large access pit to be excavated, and a staging area large enough to fuse together sections of HDPE pipe above ground. Given the congested area surrounding this section of sewer, such an excavation and staging of pipe would be problematic. Also, there was uncertainty with respect to the pressure and vibration associated with pipe bursting, and if such operations could potentially impact the railroad tracks. For these reasons, this alternative was not considered advantageous and ultimately dismissed.

Traditional sliplining of the existing sewer with HDPE pipe was also considered. The annular space between the existing sewer and the new HDPE pipe would have been filled with lightweight flowable fill. Similar to pipe bursting, this alternative was feasible because there were no sewer services located on the pipeline. However, this alternative also required access pits to be excavated, and an above-ground staging to fuse together sections of HDPE pipe. Given the congested surrounding area, this alternative was similarly dismissed.

The final alternative considered was a derivative of traditional sliplining. Specifically, this alternative involved sliplining with short lengths of mechanically joinable pipe segments. Different types of mechanically joinable pipe segments are commercially available, varying in materials of construction and joint types. Sliplining with pipe segments is accomplished by lowering individual segments down into the manhole (upstream or downstream of the existing pipeline to be rehabilitated), inserting each segment into the pipeline, and joining each subsequent segment together. The joined pipe segments are then hydraulically pushed or winched further into the sewer, from manhole to manhole. This routine is carried out until the entire sliplining process is finished. This method allows for the sliplining process to be completed without excavation and greatly reduces the required pipe staging area. This method was readily determined to be the most advantageous because ***no excavation*** is required to install the short sections of pipe, yet similar to traditional sliplining, an entirely new pipeline would be constructed inside the existing sewer.

Construction Method

Pipe Segment Selection Process

Prior to commencing with this sliplining process, it was necessary to select the specific type of pipe segment to be utilized. Specifically, it was necessary to determine the pipe material type, joint type, segment length and diameter. After evaluating the different types of commercially available pipe segments, the Town decided to utilize ISCO Buttress-Loc HDPE pipe with screw on joints, as depicted in Figure 2: Buttress-Loc HDPE Pipe. At the time, this product presented itself as the simplest to mechanically join and install, and also exhibited superior strength characteristics. Before ordering the actual pipe segments, the diameter of the insertion manhole was measured to confirm that the individual segments would fit inside the manhole. Upon measurement, it was determined that 2 ½ foot long pipe segments would fit inside the manhole. It is noted that the pipe segments are manufactured in variable lengths, accommodating a range of manhole diameters. Also, prior to commencing with the sliplining process, the existing sewer's clear opening diameter was verified by running a mandrel through it. Based on the extent of the rock protrusion into the existing sewer, it was determined that 12-inch diameter pipe segments could be sliplined.

Figure 2: Buttress Loc HDPE Pipe
Source: ISCO Industries

Sliplining Process

The sliplining process was accomplished by lowering the individual pipe segments into the insertion manhole, as shown in Figure 3: Sliplining Process. The individual pipe segments were manually threaded together with a chain wrench. An end protector plate was used to hydraulically push joined sections of the pipe segments further into the sewer. Once the sliplined pipe reached the downstream manhole, the annular space between the existing pipeline and the new pipe was filled with lightweight, pumpable flowable fill. The entire process required only a small staging area in the immediate vicinity of the upstream and downstream manholes. The actual sliplining process was completed in two days, with no excavation, no disruption to the railroad, and minimal disruption to traffic. The total cost was $70,000 (June 2006) to rehabilitate 200 feet of pipe.

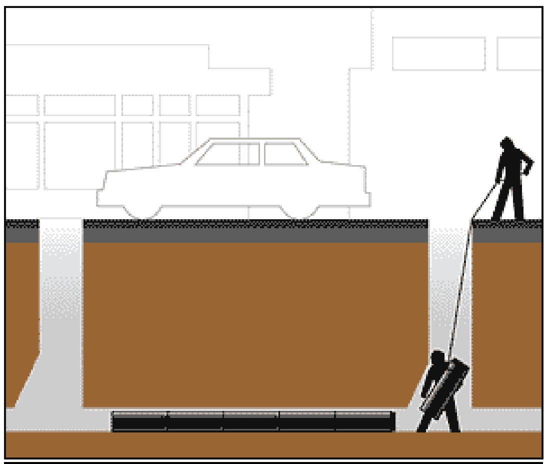

Figure 3: Sliplining Process
Source: ISCO Industries

Conclusion

This sliplining process provided the Town with a successful repair of their failing sewer located underneath the MBTA/CSX main line railroad and Route 135. This sliplining process was identified and selected by the Town after performing a detailed alternatives analysis. The process utilized short lengths of mechanically joinable pipe segments, allowing the work to be completed expeditiously and *without* excavation, and greatly reduced the required above ground staging area. As such, this rehabilitation method fully met the challenging needs of the project; providing a repair that did not jeopardize or disrupt the operations of the railroad, and did not significantly interfere with the vehicular and pedestrian traffic associated with Route 135 and the surrounding downtown area.

In summary, this sliplining process allowed the Town of Framingham to successfully complete the Howard Street Sewer Project, in the face of very significant challenges. And with the completion of this project, the Town now has the necessary sewer infrastructure improvements and associated flow capacity to accommodate their important downtown redevelopment plans.

Successful Rehabilitation Project
Utilizes Multiple Methods from the Trenchless Toolbox
Case Study of an Infrastructure Renewal Program

Joseph A. Strauch, P.E. [1]
Jeremy S. Miller [1]

Abstract

Hampden Township (Township) is a growing municipality in central Pennsylvania. Although Pennsylvania as a whole has had a stagnant population in recent years, portions of the central part of the state are growing at a brisk rate. As with many New England and Mid-Atlantic communities, the aging sewer infrastructure is not able to withstand these growing pains adequately.

The Township has been actively implementing a sewer system inspection and rehabilitation program to address the infrastructure deficiencies. Deteriorating sewer segments were identified in two distinct neighborhoods. Design documents were prepared to allow utilization of various trenchless pipe and manhole repair methods. Cured-in-place and fold and form alternatives were specified for sewer pipe rehabilitation. Epoxy and cured-in-place methods were specified for manhole rehabilitation. A case study highlighting the lessons learned during construction of the project will be discussed. Issues that had to be dealt with during construction included concrete in a portion of the existing sewer and excessive infiltration. This presentation will include an overview of the planning, investigation, design and rehabilitation construction component of the project.

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Introduction

Hampden Township is a growing municipality located in south-central Pennsylvania. The Township, which covers approximately 17 square miles (43 km^2), has a population of approximately 25,000 persons. More than 80% of the geographic area and 95% of the developed parcels within the Township are served by public sewers owned by the Hampden Township Sewer Authority. The Hampden Township sewer system was originally constructed in the early 1970s. Although numerous expansions and upgrades have been undertaken since the 1970s, a large portion of the original infrastructure remains in use. The Hampden Township sewer system consists of approximately 150 miles (240 km) of gravity sewers ranging in size from 8 to 30-inch (200 to 750 mm) in diameter, 23 pumping stations ranging in capacity from 0.10 to 6.85 mgd (0.4 to 26 Mega liters (Ml)/day), and two wastewater treatment plants (WWTP), the Pinebrook WWTP, 1.76 mgd (6.7 Ml/day), and the Roth Lane WWTP, 4.65 mgd (18 Ml/day).

In 2005 the Township completed an Act 537 Sewage Facility Plan (Plan) which identified nearly $40,000,000 in upgrades to the Hampden Township sewer system over the next 20 years. A significant portion of required upgrades included increased capacity for existing facilities to handle peak wet weather flows. A flow meter study completed as part of the Plan indicated that portions of the Township's sewer system are subject to significant infiltration and inflow (I/I). The Plan, therefore, included an I/I reduction program as one of its chosen alternatives in order to reduce capacity needs.

Goals of the I/I reduction program included inspection of the entire sewer system within the 20 year planning period and 50% removal of I/I from the sewer system on an annual average flow basis. These goals require an average inspection of 3,000 feet (900 meters) of sewer per month and the removal of approximately 250,000 gpd (950 m^3/d) of I/I on annual average flow basis.

In order to help identify problem areas, the Township was divided into 28 separate subbasins. The Plan prioritized the subbasins with regards to total flow per equivalent dwelling unit (EDU), rainfall derived I/I (RDII) per inch of rain, and RDII per linear foot of sewer. The six highest rated subbasins were included in a pilot study area. As part of the pilot study, the six highest priority subbasins were to be inspected and rehabilitated in a five year period.

Inspection activities included televising and air testing all sewers, and physical inspection of all manholes, within the six priority subbasins. The Township currently owns and operates two TV trucks (both equipped with WINCAM software) which were used to complete all air testing and televising activities. The WINCAN software has incorporated the NASSCO (National Association of Sewer Service Companies) rating system for prioritizing sewer line defects. All defects observed during the TV and manhole inspections are recorded in the WINCAN software and prioritized in accordance with the NASSCO rating system.

Rehabilitation activities were separated into categories, In-house repairs and contracted repairs. In house repairs include grouting, small dig-ups and pipe replacements, and internal spot lining repairs. All large pipe replacement and lining projects are put out for bid.

NASCCO Pipeline Assessment and Certification Program

The Township and many other communities are beginning to utilize asset management and condition assessment programs to assist in the overall utility management process. A key program for assessing wastewater collection systems is the NASSCO Pipeline Assessment and Certification Program (PACP). The program is a standardized format to evaluate internal television inspections of sewerline assets. Visual defects are identified utilizing key structural and O&M parameters. Standard defect codings are the primary components of the program. Collection system assessment through PACP, or other similar methods, can then be recorded and evaluated.

The condition of the sewer collection system assets (pipelines and manholes) is based upon the rate of deterioration of the originally installed materials. Deterioration factors include, but are not limited to, soil conditions, groundwater, hydraulics, surface loadings, installation and internal conditions (roots, grease, surcharging, etc.). Pipeline defects such as deformation, subsidence or hydrogen sulfide attack are subject to become more severe over time.

Televised sewers are categorized under PACP using four categories of coding; (1) structural defects, (2) operational/maintenance defects, (3) construction features and (4) other.

Structural defects are separated into twelve different groups – crack, fracture, broken, hole, deformed pipe, collapse, joint defects, surface damage, lining defect, weld failure, point repair, and brickwork. Operational/maintenance defects are separated into five different groups – deposits, roots, infiltration, obstruction, and vermin. Construction features are separated into four different groups – tap, intruding sealing material, line, and access points. The other category includes miscellaneous observations.

There are two main classifications of defect coding. The first are point defects which occur at a discreet location, and are coded individually. The second are continuous defects which are the same defect that occurs over a length of sewer.

Annual recording and reporting of PACP results can be used to prioritize future rehabilitation efforts. Initial television inspections are a baseline for condition assessment of those assets. Future inspections can be easily compared to historical information, and specific deficiencies can be analyzed to see how the severity of the defects has progressed over time.

The Pilot Study

Early in the inspection of the pilot study area, two significant sources of I/I were identified. The first section (Section A) consisted of approximately 1,300 feet (400 meters) of asbestos cement sewers ranging in diameter from 18 to 24 inches (450 to 600mm). This portion of sewers, which spanned 8 manhole runs, contained numerous defects including cracked piped, leaking joints, and severe spalling. The 8 manholes along Section A also exhibited significant deterioration and required rehabilitation. (See Figures 1 and 2) Weir readings indicated that the defects contained in Section A contributed approximately 65,000 gpd (250 m^3/d) I/I to the sewer system on an annual average flow basis.

The second portion of sewers (Section B) consisted of approximately 2,100 feet (640 meters) of 8-inch (200 mm) asbestos cement pipe spanning 7 manhole runs. Defects identified in Section B were similar in nature to the defects found in Section A. Weir readings indicated that the defects contained in Section B contributed approximately 18,000 gpd (70 m^3/d) of I/I to the sewer system on an annual average flow basis.

Figure 1
Interior – Deteriorated Manhole

Figure 2
Exterior – Deteriorated Manhole

Based upon an evaluation of the required pipe and manhole defects, it was determined that trenchless rehabilitation would be the most cost-effective alternative. The rehabilitation of Section A and Section B was combined into one project in order to reduce unit costs. The total project included the lining of approximately 3,400 feet (1,000 meters) of pipe. A combination of fold and form and cured in place lining was used. Internal rehabilitation was also the chosen alternative for the manhole rehabilitation. An epoxy lining was used to rehabilitate six of the 8 manholes. Cured in place lining was used on the remaining 2 manholes.

Design and Bid Phase

During the design, the technical specifications were developed for the Township and included a variety of trenchless methods to ensure a cost-effective and efficient solution. Fold and Form and Cured-in-place sewer lining techniques were included in the specifications. Cured-in-place manhole lining was also part of the technical design for two of the severely deteriorated manholes. Epoxy coating of other manholes was also included in the project.

Preliminary opinions of probable cost for the entire project were estimated to be on the order of $250,000. Four bids were received ranging from $238,000 to $300,000. Bid packages were reviewed and the project was awarded to the lowest responsive and responsible bidder.

The selected contractor utilized various trenchless methods to complete the rehabilitation project. The larger diameter sewers [1,300 linear feet (400m) of existing 18 to 24-inch (450 to 600mm) diameter], were rehabilitated utilizing Cured-in-place materials. The smaller diameter sewers [2,100 linear feet (640m) of existing 8-inch (200 mm) diameter], were rehabilitated utilizing Fold and Form materials. Manholes were rehabilitated utilizing epoxy coating and Cured-in-place lining (See Figures 3 and 4).

Figure 3
Cured-in-Place Manhole Liner

Figure 4
Installation of Cured-in-Place Manhole Liner

Conclusions and Lessons Learned

The project removed an estimated 83,000 gpd (315 m^3/d) of I/I from the Hampden Township sewer system on annual average daily flow basis while extending the life of the rehabilitated sewer sections up to an estimated 50 years.

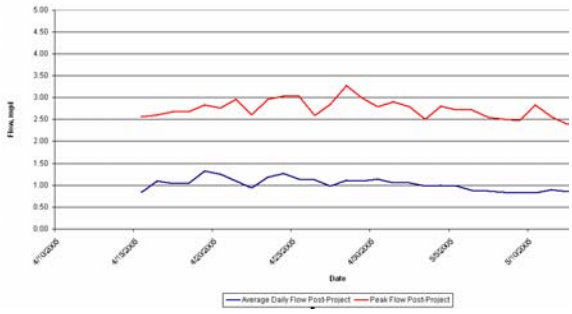
Figures 5 and 6 show a graphical representation of the pre and post-project flows for a segment of Section A. The figures show a comparison of average daily and peak flows. Post-project flows are significantly lower after sewer system rehabilitation.

The project costs for lining the sewers sections was more than 50% less than the estimated costs to replace the sewers. The Township, therefore, saved more than $200,000 by lining the existing pipe and manholes.

Figure 5

MH No. 1204/1203 - Sears Run Right-of-Way
Pre-Project

Figure 6



MH No. 1204/1203 - Sears Run Right-of-Way
Post-Project

The original cost for the project was approximately $238,000. Two change orders were requested by the contractor during construction. The contractor requested additional payment for the following items:

- Concrete removal in pipe ($4,700)
- Excessive grouting prior to lining ($7,000)

One segment of the larger diameter (20-inch (500mm)) sewers in the project contained concrete of unknown origin which had hardened in the bottom of the pipe. The contractor went to great lengths to remove the concrete prior to lining, and the Township compensated him with a payment of $4,700 based on time/materials basis.

The contractor also requested additional payment for performing what he claimed was excessive grouting at pipe joints for the purpose of stopping active infiltration prior to lining the pipe at a stream crossing. The Township denied this claim for additional payment based upon the requirements of the technical specifications for the project.

The approved change order in the amount of $4,700 equated to less that 2% of the original contract value.

The overall project was very successful for the Township. Competitive bid prices for rehabilitation based upon a variety of trenchless methodologies, coupled with a significant reduction in I/I, is a beneficial start to the Township's long range goals.

References

Related Material

Strauch, Joseph A. and Wetzel, Dawn M.; *"Collection System Asset Management – Benchmarking Performance Through Annual Reporting";* American Society of Civil Engineers Pipelines 2006 Conference, Chicago, Illinois, July - August 2006

Related Websites

National Association of Sewer Service Companies (NASSCO)
Pipeline Assessment and Certification Program (PACP)
http://www.nassco.org/pacp.html

TURN-KEY CONDITION ASSESSMENT AND REHABILITATION/REPLACEMENT SOLUTION FOR AN EFFLUENT FORCE MAIN

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Abstract

The City of West Palm Beach constructed approximately 6 miles of effluent force mains in 1974 extending from 23rd Street using 42" Pre-stressed Concrete Cylinder Pipe (PCCP) and from Congress Ave using 48" PCCP to the East Central Regional Wastewater Treatment Plant within the City. The force mains normally run below the prevailing water table or are subject to wetting and drying cycles and are generally now below multi-lane paved surfaces. The City had experienced failures and wanted a complete investigation into the cause and extent of any possible deterioration/distress while the force main remained in service. The City also wanted to get recommendations for a possible rehabilitation and replacement program.

The Pressure Pipe Inspection Company (PPIC) led a team comprising of Jordan, Jones and Goulding (JJG), Jason Consultants and an excavation contractor to provide the City with a complete turn-key solution. This paper will discuss the methods used to determine the existing condition of the force main: Acoustic emission testing, historic H_2S readings, chemical dosing records, repair/maintenance records, local soil conditions, hydraulic model runs, as well as the stress and strain in the PCCP elements. Additionally, it will cover the recommendations made to the City for a rehabilitation and replacement program, and highlight the advantages of a turn-key solution.

Introduction

The City of West Palm Beach owns and operates over 600 miles of water and 400 miles of waste water pipelines. The City's Public Utilities department mission statement includes:

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> "To develop long range strategic plans to meet future infrastructure and utility service needs for community growth, development and expansion."

The first part of developing a long range strategic plan for a pipeline is to determine the existing condition of the line. Once the condition is known, the amount of useful life can be estimated and informed decisions as to replace or repair certain sections or all of the line can be made.

The 48"/42" effluent force main (F.M.) is of particular importance to the City as it is the only force main that transports effluent from both the town of Palm Beach and the City of West Palm Beach to the East Central Regional Waste Water Treatment Plant. The City's original plan in the 70s was to build a parallel line that would take treated effluent form the plant and could act as a redundant line; however, it was never built. Additionally, past failures of sewer force mains in the area allowed effluent to get into nearby canals, which make there way to the ocean resulting in several miles of beach closures. Do to the lack of redundancy, and the consequences of failure, the City is taking a proactive approach to the management of this line.

The City has performed routine maintenance on the air release valves (ARV) but does not otherwise know the condition of the line. The City desires a complete investigation into the causes and extent of any possible deterioration of the 42"/48" F.M. and ways to initiate non-destructive repairs to the force main while it remains in service. This falls in line with the Public Utilities mission to develop long range strategic plans.

The Pressure Pipe Inspection Company (PPIC) was approached by the City to provide the condition assessment solution. PPIC teamed up with engineering firms Jordan, Jones and Goulding, Jason Consultants and a local contractor to provide a turnkey solution for the City.

The overall solution for the City includes acoustic monitoring of the entire pipeline performed by PPIC, an existing conditions report by Jason Consultants and a feasibility report stating a repair and replacement plan by Jordan, Jones and Goulding. The proposed solution will take seven months.

42"/48" Effluent Force Main

The PCCP pipe supplied for the City of West Palm Beach's 42"/48" F.M. was made by Price Brothers. According to the Pipe Specification, Project No. 50.74H-1, dated 6/26/74, the pipe was lined cylinder pipe. Normally PCCP pipe supplied in these diameters is embedded cylinder pipe.

PCCP pipe is constructed by first casting a thin steel cylinder inside or outside of a concrete core. With lined cylinder PCCP, the concrete core is cast inside the steel cylinder (Figure 1.0). The concrete core is then reinforced by spirally wrapping high strength wire (prestressing wire) around the steel cylinder. The prestressing wire is then coated with a mortar, which is intended to provide corrosion protection to the wire. The prestressing wire keeps the concrete core under compression under normal operating conditions.

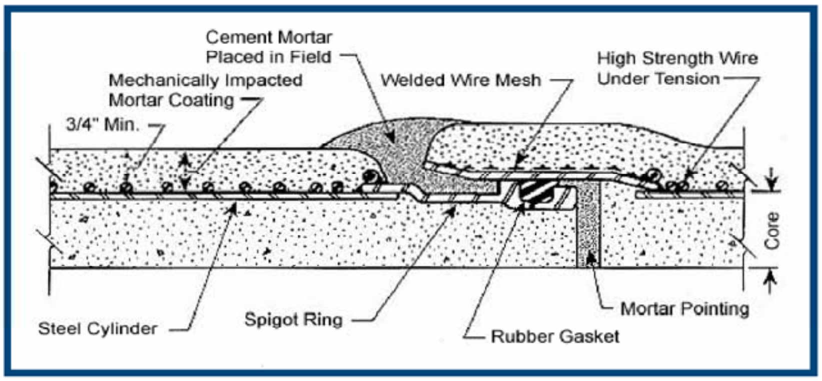

Figure 1: Lined Cylinder PCCP Pipe

Each of the components shown in the above figure is designed and manufactured with a specific purpose in mind. PCCP is a durable, resilient pipe material as long as all the components remain intact to fulfill their intended purpose. The failure of any one of the components may lead to deterioration and the subsequent failure of the pipeline.

The force main runs through the City of West Palm Beach, under roads, easements beside canals, a park, a golf course and goes from STA 190+49 to STA 495+32.

Past records for the force main showed that two new plug valves were installed in 1995, one near STA 482+41 and the other close to the treatment plant. A plug on the end of a tee had failed near STA 486+41 due to improper seating of the plug. An ARV had failed due to corrosion dumping approximately 3,000,000 gallons of effluent into nearby canals. Routine maintenance and operation of the ARVs had been performed on 10 of the 23 ARVs on the force main. The 13 ARVs that can't be serviced are locked in an open position and it is unknown if they still work. There have been no catastrophic failures or ruptures of the PCCP pipe itself.

Existing Condition

The existing condition of the 42"/48" F.M. will be determined by combining the information gained by walking the line, visual inspections of the air valves, and analysis of specific pipeline details with current loading, maintenance records and the results of the acoustic monitoring.

Air Release Valves

The line was walked and a visual inspection of the air release valves (ARVs) was performed on December 11th and 12th, 2006. The visual inspection confirmed that several ARVs were not operable and proposed a risk to the structure of the pipeline. Air release valves allow air and other gases to escape the pipeline while keeping the fluid inside. When an ARV does not operate properly gasses become trapped in the pipeline. For effluent mains H_2S gas can build up and increase the rate of corrosion of both the air valve and the PCCP pipe. The City will be given a solution to replace the inoperable ARVs while the main is in service.

Picture 1: Typical Air Valve

Since ARVs are often a source of odor as well, some of the devices have been closed to reduce the level of complaints. Because these devices are located at high points in the line to maximize the removal of entrained gases, closed ARVs, particularly in low velocity mains, allow gas bubbles to stay trapped further exposing the interior of the pipe to corrosion. In coastal areas, H_2S measurements, especially in flat collection and pressure lines not flowing full, are typically found at between 300 and 800 ppm in air. Submerged and removed ARVs present the same problem, allowing high concentrations of sulfides to linger particularly around the crown of the pipe. Exposed metal substrate and pre-stressing wires or bars that see these levels of H_2S deteriorate at a much faster rate than comparable exposure in systems with higher velocities and generally lower sulfide concentrations. The size, location, physical arrangement (piping) and volume displacement of ARVs was reviewed to assess whether existing ARV deployment was adequate for current and future flows in the line.

Pipeline Design Criteria

The following table summarizes the important design requirements and relevant pipe material specifications for this project.

Table 1. Pre-stressed Concrete Cylinder Pipe (PCCP) Design - 50.74H-1

Nominal Diameter, inch	42	42	48	48
Type of Pipe	SP-5	SP-5	SP-5	SP-5
Design Requirements				
Cover Depth, ft	10	20	8	24
Internal Pressure, psi	100	100	100	100
Test Pressure (6 hr), psi	150	150	150	150
External 3-Edge Dead, lb/ft	4395	8915	3780	12162
Pipe Specifications				
Core Thickness, inch	2-5/8	2-5/8	3	3
Minimum Compressive Strength at Wrapping, psi	3000	3580	3000	4360

Nominal Diameter, inch	42	42	48	48
Min. Coating Thickness (over wire), inch	5/8	5/8	5/8	5/8
Cylinder Thickness, inch	0.0598 (#16)	0.0598 (#16)	0.0598 (#16)	0.0598 (#16)
Cylinder Yield Strength, psi	27,000	27,000	27,000	27,000
Wire Diameter, inch	0.162 (#8)	0.162 (#8)	0.162 (#8)	0.192 (#6)
Class of Wire	II	II	II	III
No. of Wraps per ft.	16.67	30.00	18.46	27.27
Minimum Tensile Strength of Wire, psi	231,000	231,000	231,000	252,000
Po- Zero Compression, psi	157	274	156	334
Pb- Minimum Bursting, psi	390	619	370	714

In comparison to the latest specification for PCCP pipe, AWWA C301-99, the above pipe material specifications would not conform in four specific areas. The minimum gage thickness for the high strength pre-stressing wire is now #6, or a minimum wire diameter or 0.192 inches. Only the 48-inch pipe designed for 24 ft. of cover complies with this requirement. The other pipe designs incorporate #8 gage wire with a diameter of 0.162 inches. Second, the minimum compressive strength of the concrete core required at wrapping is now 3,500 psi. Two of the pipe designs, namely the 42-inch for 10 ft. of cover and the 48-inch for 8 ft. of cover, have a concrete core with only a 3,000 psi compressive strength at wrapping. Third, the minimum coverage of mortar over the pre-stressing wire should be ¾ inch, not 5/8 inch. And lastly, the minimum yield strength of the steel cylinder is now 33,000 psi. Assuming a yield strength of 27,000 psi, which is the minimum for some PCCP pipes supplied during this same time period, as no value was included in the Price Brothers specification. Needless to say, all the pipes did meet the specification in effect at the time of the project, which would have been AWWA C301-72.

Structural Analysis

Concurrent with the change in the AWWA C301 standard in 1992, AWWA C304 was developed to provide a more rigorous and accurate design method for PCCP pipe. In 1999 the latest version of AWWA C304 was issued and is the current version used by the industry. The standard is a very good tool for checking the adequacy of the design of pipes supplied in earlier years, whose design was based on the state of the art at the time – mainly the cubic parabola method.

The new AWWA C304 design approach can be very helpful in checking the risk of failure of existing pipes. Rather than using the original loading conditions, the current loading conditions should be investigated and used in the analysis. We find very often that pipes were often designed for no traffic loading, but now run down the middle of major roadways given unexpected growth and direction in some communities. This is the case for the PCCP force main in question. During a recent walk over of the line, it was observed that 85% of the force main is now located under heavily traveled road surfaces. The original design pressures were conservatively selected based on information in the scope. However, no provision for transient pressure was apparently included.

The new design method incorporates the concept of "limit states" into the design approach. There are three limit states defined, against which various criteria are calculated and compared. The three limit states and their C304 definition are:

1. Serviceability – ensure performance under service load
2. Elastic – onset of material nonlinearity
3. Strength – safety under extreme loads

Based on the original design requirements, three of the pipe designs would not conform to the new AWWA C304-99 limit state requirements. Two of the non-conforming pipe designs are at the deepest burials, namely the 42-inch diameter at 20 feet and the 48-inch diameter at 24 feet, and the third is the 42-inch diameter at 10 feet. Although three of the pipe designs failed to meet some of the serviceability and elastic limit state requirements in AWWA C304-99, none of the pipes failed to meet the strength requirements.

Sensitivity to Wire Breaks

The sensitivity of each pipe design to the affect of wire breaks can be examined with a non-linear finite element analysis (FEA), and then the results properly calibrated with actual tests on the same pipe design. The normal assumption in a FEA is that all wire breaks are contiguous, i.e. they are in the same circumferential position along the pipe axis. Naturally if the cause of the wire break is from exposure to the elements as a result of localized damage to the mortar coating, then it would be reasonable to expect contiguous wire breaks. However, in their calibration work PPIC has found many wire breaks are not contiguous. It takes approximately 3 inches of embedment in the dense mortar coating for the wire to fully develop bond, so the impact of a break in a single wire doesn't negate all of the pre-stressing force in the same circumferential plane. Consequently, the impact of say 30 contiguous wire breaks is more critical than 30 adjacent wire breaks distributed around the circumference.

Jason Consultants has carried out risk assessments on a number of major PCCP projects. In terms of the structural integrity of the PCCP pipe based on the AWWA C304 analysis, they have used the following severity ranking to prioritize pipes for closer inspection:

1 – no issues, microcracking limits are not exceeded
2 – microcracking limits are exceeded, but elastic limits are not exceeded
3 – visual cracking limits are exceeded, but not elastic limits
4 – elastic limits are exceeded, but strength limits are not exceeded
5 – strength limits are exceeded

Another insightful way of examining the impact of wire breaks on the pipe's structural performance, is to "artificially" break wires in the AWWA C304 analysis method by gradually reducing the amount of prestress wire. Wire reductions of 10, 20, 30 and 40 percent will usually demonstrate how sensitive a pipe design can be to wire breaks. Jason Consultants has seen pipes with only a 15% reduction in wire go from a severity rating of 2 all the way down to 5, and the factor of safety dropping to below 1.0. For other designs, the impact has been negligible. As a first step, this type of analysis will be carried out on the four pipe designs to see which ones are most sensitive to the affect of wire breaks. This information, coupled with the results of PPIC's acoustic monitoring, will determine which pipes if any fall into a high failure risk category.

Acoustic Monitoring

Acoustic Emissions Testing (AET) will be performed between February, 2006 and June, 2006. During an AET inspection acoustic sensors are placed along the pipeline and the energy that is released when a prestressing wire breaks or slips is recorded and the origin location is calculated. Wire breaks and slips both indicate the presence of distress and loss of prestress and are refereed to as wire related events (WRE). Locating which of the PCCP pipes, if any, that are actively deteriorating is a key step in determining the condition of the main.

The design of an AET inspection depends on a lot of factors including pipe type, pipe diameter, pipeline pressure and overall objective of the test. In this project the objective is to find any pipes that are in later stages of distress. At this point it is unknown if there are any distressed pipes in the main. PPIC has designed a cost effective AET to first sweep over the pipeline to determine if there are any areas in the main that are deteriorating and then second to focus in on any areas that show signs of distress. The initial spacing between sensors will be close to 600 ft and the entire main will be monitored for a minimum of 700 hrs. In areas where WRE are detected the spacing may be reduced to 300 ft and the monitoring time increased if necessary to maintain a proper assessment of the pipe.

Each monitoring site will include an AH5 data acquisition unit, an accelerometer acoustic sensor, two 12 V batteries and multiple solar panels. The accelerometer will be mounted to the surface of the pipe using epoxy. Each site will take approximately half a day to set up which includes exposing the pipe surface, mounting the sensor, backfilling, temporary paving and hooking up the electronics. The test will begin with 16 sites and then after 700 hr the sites will be rolled over to the next 16 sites. This will be repeated until all 54 sites along the line have been monitored.

Challenges to performing AET on this particular force main include the effluent fluid, the interior coating, pipes below the water table and pipes under roadways. The effluent and the interior coating will acoustically dampen the acoustic signal received at the acoustic sensor. In a previous AET inspection of an effluent main in Florida the listening distance between sensors had been roughly half of what is typically seen in water pipelines. This damping effect will be unique to each force main and will be accounted for during the test by reducing the sensor spacing in distressed areas.

Where the pipeline is under the water table special considerations need to be made. The first step is to well point the area where the pipeline is to be exposed. This involves drilling several small holes around the dig spot and then pumping the ground water away. The pumps will be turned on several hours before digging to lower the ground water level. This allows the excavation crew to dig down to the pipeline and create a clean dry surface to mount the accelerometer. After the accelerometer is installed it is enclosed in a water proof housing to account for the ground water.

Picture 2: Well Pointing

Picture 3: Accelerometer w Housing (no lid)

When the pipeline goes under roads it will generally require traffic control, dig permits and re-paving of the surface to install an accelerometer. To avoid this PPIC plans to use air valve chambers where the force main can be easily and safely exposed. The old chambers on the force main don't have a concrete bottom, therefore the pipe surface is sometimes already exposed or under only a foot of gravel. Each chamber will be probed and then cleaned out to asses if enough of the pipe surface can be exposed to install an accelerometer.

The AET results can then be directly compared to the theoretical structural analysis of the pipe. It is expected that the majority of WRE will occur in high risk areas that were pointed out in the structural analysis.

Feasibility Report

After combining the results of the AET and the theoretical analysis to form the existing conditions report, JJG will conduct a workshop with the City and the project team to discuss repair/replacement/renovation options to look at:

a) a parallel force main (or construction of originally planned "future 48" outfall main for treated effluent)
b) repair options and or
c) renovation options with focus on trenchless methods.

From the workshop a feasibility report will be prepared to analyze costs to rehabilitate line segments based on the priorities identified in the existing conditions report.

It is anticipated that non working air release valves will need to be replaced and a plan to repair distressed pipes will need to be given highest priority. The redundant force main will be looked at both as an overall safety and capacity concern as well as incrementally at locations where the probability of spills will create higher risk for the City. Such locations will include the main's crossing beneath interstate highway I-95 and at several of the line's aerial crossings over canals and drainage waterways. Parallel lines in these spots will provide a back-up route to minimize loss of untreated flow in the event of planned repair or sudden failure. Redundant lines would also facilitate "renewal" of the existing main via slip-lining or cured-in-place methods and replacement of pipe where needed, especially on aboveground segments. One aspect of preparing for sudden failures is having the City equip itself with "repair kits" including pipe, fittings, valves, dewatering and bypass pumps, vactor trucks, etc. and identifying crews and contractors to make repairs as part of an overall Emergency Response Plan.

Conclusion

The City of West Palm Beach requires an assessment of their 42"/48" Effluent Force Main while the line remains in service. The project team of PPIC, Jason Consultants and Jordan Jones and Goulding have provided a solution to the City that will give the information they need to effectively make a long range plan for the force main. The results will be obtained without affecting the operation of the force main and with limited resources from the City therefore the City can concentrate on operating the pipeline.

PCCP Inspection: Prioritizing Risk, Assessing Shutdown Impacts, and Executing the Inspection

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Abstract

Responding to worldwide failures of prestressed concrete cylinder pipelines (PCCP), the City of Phoenix initiated a program to inspect its entire 150-mile system of PCCP water transmission mains over the next seven years. This system serves 1.4 million people over more than 500 square miles and can only be inspected during low demand winter months. Most of the system has never been taken out of service.

The inspection program began with development of a masterplan to prioritize each pipeline for inspection. The overall risk associated with the failure of each pipeline was assessed using readily-available data. The risk considered:

- Pipeline diameter to predict the number of affected customers
- Nearby land use as a tool to assess potential property damage due to flooding
- Pipeline age to reveal quality of construction methods and materials
- Corrosive soil and groundwater conditions near the pipeline

This paper demonstrates the use of a risk-based methodology to prioritize inspections, the use of hydraulic modeling to predict and address shutdown impacts, and shutdown planning. Each step of the process is related to a 4-mile, 66-inch diameter pipeline in Phoenix. This 40-year-old pipeline serves a large portion of west and central Phoenix.

Introduction

The water system of the City of Phoenix (City) contains over 200 miles of large-diameter pipe, with diameters ranging between 42 inches and 108 inches. PCCP comprises over 150 miles of this pipe. The City initiated a proactive program to identify and respond to structural deterioration in its water system by developing a masterplan to inspect all of its PCCP pipelines. The primary goal of the masterplan was to minimize the risk of rupture of a PCCP pipeline by inspecting pipelines in the shortest time possible. Secondary, but also extremely important goals included minimizing service impacts to water customers and maintaining the City's already extensive plant and pipeline maintenance and capital improvement schedule.

Shortly following completion of the masterplan the City inspected a 4-mile, 66-inch diameter pipeline which serves a large portion of west and central Phoenix.

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Although the risk analysis determined the pipeline to have about an average inspection priority, the City decided to inspect the pipeline in January 2007 because it was removed from service concurrent with a nearby water treatment plant maintenance shutdown. This case-study pipeline is used to illustrate the risk assessment applied to the City's PCCP pipelines.

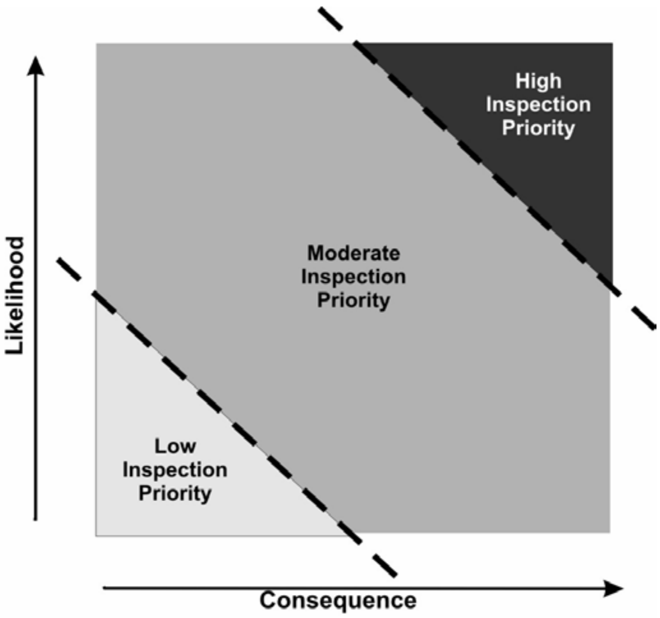

Figure 1. Relative Risk Assessment

Risk Assessment

The City began its program by first inspecting those pipelines at highest risk of failure. This targeted approach lessened the risk of a pipeline failure during the program by ensuring that the most critical pipelines were inspected first. Masterplan development began by prioritizing the pipelines for inspection according to their *relative* risk of failure.

A traditional risk assessment was applied to the City's PCCP pipelines. This approach identifies the consequence and likelihood of a particular pipeline failing relative to other pipelines in the system (Bell, 2001). Pipelines with both a high consequence and high likelihood of failure are inspected at the beginning of the program. Conversely, pipelines with both a low consequence and low likelihood of failure are inspected at the end of the program. The traditional risk assessment approach is described in Figure 1.

The risk assessment began by dividing the City's 150 miles of PCCP into 42 pipelines, three to five miles in length, based on system operation and facility constraints:

- Valve locations. In-line valves are needed to isolate the pipeline at the inspection limits
- Nearby pump stations and pressure valves. Some facilities cannot be removed from service without significant capital expenditures, such as additional pipelines or pump stations to supply water to affected customers
- Pressure zone served. Limiting the number of pressure zones affected by each shutdown to one lessens the impact of shutting down a pipeline.
- Installation date. Inspection priority is heavily dependent on installation date, as discussed below.

Factors Considered in Likelihood of Failure. This assessment did not attempt to define the precise probability of a pipeline failure. The purpose of this evaluation was to assess the likelihood of a particular pipeline failing compared to other pipelines in the water system. A large number of factors contribute to pipeline failure. Because the purpose of this risk assessment was to prioritize each of the City's 42 pipelines for inspection within the shortest reasonable time, only readily available data was considered. This data included the pipeline's installation date and nearby soil corrosivity.

Installation Date. The sensitivity of PCCP to failure based on installation date is largely related to the history of prestressing wire manufacture. The manufacturing methods and material used in the production of prestressing wire have evolved significantly since it was first manufactured in the early 1940's. In 1954, when PCCP was first used in the City of Phoenix, PCCP was manufactured to AWWA C301-52, Standard Specifications for Reinforced Concrete Water Pipe – Steel Cylinder Type, Prestressed and ASTM A227-47, Standard Specifications for Hard-Drawn Steel Spring Wire. Changes to AWWA C301 and ASTM A227 in the mid 1960's permitted the use of higher tensile strength wire, specifically Class III, 8-gage wire which ultimately proved more susceptible to corrosion-induced brittle failures. Revisions to PCCP manufacturing and testing standards to improve wire quality began in 1984 and were essentially complete in 1988 (Lewis, 2002). For the relative risk assessment, the City of Phoenix pipelines identified as most susceptible to corrosion-induced brittle failures are those installed between 1968 and 1984. Pipelines installed between 1984 and 1988 were assigned a lower relative failure likelihood. The next lowest risk was assigned to pipelines installed before 1964, while the lowest relative risk was assigned to pipelines installed after 1988. Although the most conservative approach would seem to be inspecting all pipelines installed between 1968 and 1984 as soon as possible, more than 40 percent, or 64 miles of the City's pipe, were installed during these years. As the City's inspection budget and schedule required these inspections be scheduled over several years, further factors were needed to refine the inspection prioritization.

Proximity to Waterways and Soil Corrosivity. In arid and semi-arid environments, such as those found in the City of Phoenix, close proximity to waterways may subject pipelines to alternate wetting and drying conditions. Alternate wetting and drying tends to concentrate chlorides and oxygen in the pores and capillaries of PCCP's mortar coating (Benedict, 1999). The mortar coating is a highly alkaline cement paste which passivates prestressing wire by forming a thin film of gamma ferric oxide on the metal surface. Chloride ions initiate and sustain corrosion of prestressing wires by penetrating this film (Price, 1998). Since corrosive soils, such as those high in chlorides, combine with alternate wetting and drying cycles to initiate and sustain PCCP corrosion, soil corrosivity was considered only for pipelines which might reasonably experience fluctuating groundwater levels in the pipe zone. A 500-foot maximum distance criterion was established based on historical groundwater data collected in the vicinity of other City of Phoenix pipelines. United States Department of Agriculture Soil Surveys referenced for this project included *Maricopa County, Arizona, Central Part, STSSAID AZ651, September 25, 2002, Eastern Maricopa and Northern Pinal Counties Area, AZ655, May 30, 2002, and Aguila-Carefree, Parts of Maricopa and Pinal Counties, AZ645, December 9, 2002.*

Factors Considered in Consequence of Pipeline Failure. Relative assessment of consequence of failure does not attempt to assess the monetary cost of a pipeline failure, but rather identifies and quantifies factors which increase the cost of a failure. As with likelihood of failure, a large number of factors contribute to the cost of a pipeline failure. To lend simplicity to this assessment, only the three most significant factors are considered: adjacent land use, pipeline diameter, and proximity to major roadways and railways.

***Adjacent Land Use*.** Certain land uses carry a higher consequence of failure than others due to higher population densities and greater surface improvements. For example, commercial land use typically carries a higher consequence of failure than industrial because more people are affected and traffic densities are higher. The Maricopa Association of Governments General Plan, which outlines planned land use for the Phoenix metropolitan area, was used to identify land uses adjacent to the City's pipelines. Commercial and high-density residential areas were assigned the highest consequence of failure. Industrial and medium-density residential areas were assigned a medium consequence of failure. Unused space and parks were assigned the lowest consequence of failure.

***Pipeline Diameter*.** Replacement cost after a pipeline failure, potential damage caused by a pipeline failure, and the number of affected customers increase as pipeline diameter increases. Pipelines were grouped into three categories: 42-inch to 60-inch, greater than 60-inch but less than 84-inch, and 84-inch and greater. The smaller diameter category was defined so that it would include the majority, nearly 90 percent, of the City's pipelines. The larger diameter categories were defined such that only the 10 percent of pipelines largest in diameter would be assigned the highest priority. These larger pipelines tend to be located immediately downstream of the City's water treatment plants and generally feed the rest of the City's water system. A pipeline failure close to a water treatment plant would affect far more customers than the failure of a smaller diameter main farther from a water treatment plant.

***Proximity to Major Roadways and Railways*.** The failure of a PCCP pipeline generally results in collapse of the soils and surface improvements above and around the pipeline. The vast majority of the City's large diameter water system is located in major arterial streets. These pipelines were assigned a moderate consequence of failure. Pipelines in close proximity to a railroad or freeway were assigned a high consequence of failure. Pipelines in open areas or minor streets were assigned the lowest consequence of failure.

Non-Quantitative Factors

To simplify and expedite this assessment not all likelihood and consequence factors were considered. One noteworthy omission is pipeline operation, including operating pressure and strategies which might affect the likelihood of a pipeline to experience a surge event. Operating pressure directly affects the consequence of a pipeline failure, while operating strategies, such as in-line valve operation and pumping practices, may affect the likelihood of a failure. Operating data was not available systemwide. This factor was omitted to avoid introducing significant inaccuracy and false precision.

The Case Study

The 4.5-mile, 66-inch diameter Deer Valley South Pipeline has been selected to illustrate pipeline prioritization. Installed in 1960, this pipeline once traversed largely undeveloped land in western and central Phoenix. However, several decades of growth have brought commercial and residential development to the area.

The Deer Valley South Pipeline was selected as the case study because it was the first pipeline inspected following the risk-based prioritization.

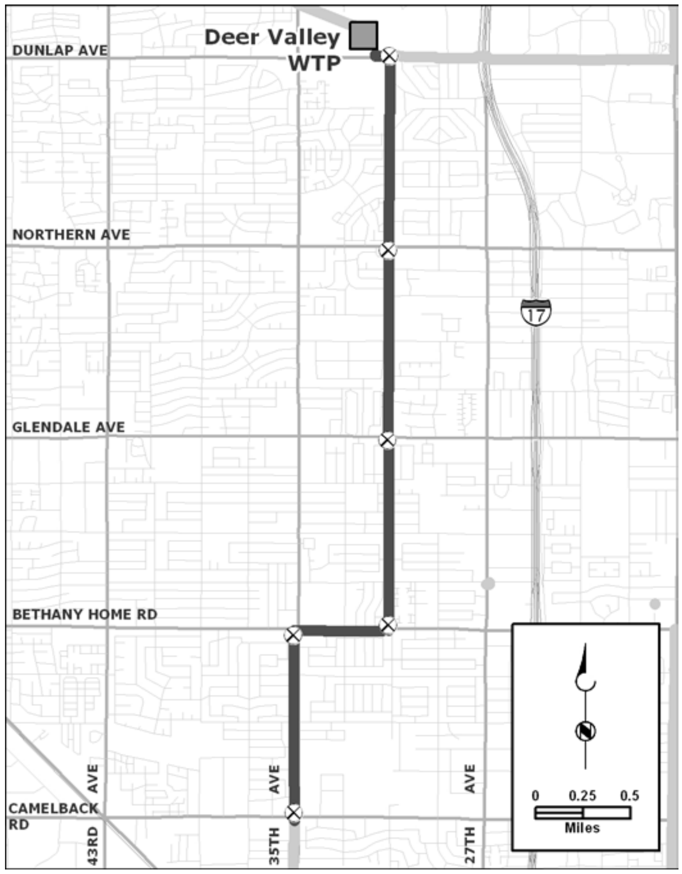

Figure 2. Deer Valley South Pipeline

Deer Valley South Pipeline Prioritization. Based on the failure likelihood and consequence factors discussed above, the Deer Valley South Pipeline was assigned an inspection priority. The City's Geographical Information System (GIS) was used to simplify data management. Within the GIS the pipelines are defined as independent objects between fittings, valves and other appurtenances. For each of these objects the City's GIS contains location, installation date, and pipeline diameter.

The waterway locations, soils, and land use data was available in files compatible with the City's GIS. The GIS facilitated spatial querying of the pipeline location, installation date, diameter, waterway location, soils and land use data to determine relative likelihood and consequence of failure of each object. The relative likelihood and consequence of failure of each pipeline was a length-weighted average of the individual objects comprising the pipeline. Figure 3 demonstrates determination of failure likelihood and consequence for a 400-ft section of the Deer Valley South Pipeline.

The pipeline object shown in Figure 3 was assigned a moderate likelihood of failure. Because the drainage ditch to the north is within 500 feet of the pipeline segment, and may subject the segment to a cyclical wetting and drying environment, the aggressive nature of the Gilman Loam-type soil to PCCP was considered. This was tempered, however, by the pipeline installation date. Installed in 1960, this pipeline is not expected to have the higher tensile strength, smaller gauge wire present in pipelines installed between 1968 and 1984.

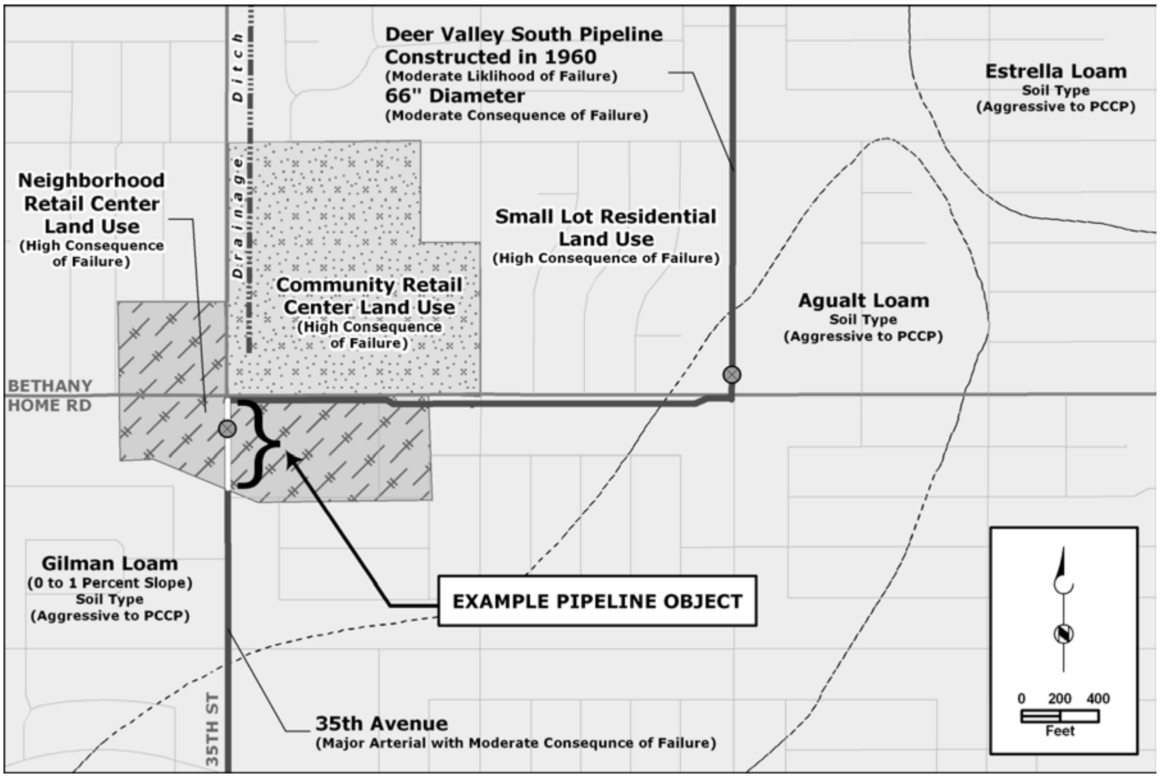

Figure 3. GIS-Based Spatial Query for Risk Assessment

The pipeline object shown in Figure 3 was assigned a moderate consequence of failure. Proximity to land designated as a neighborhood retail center gave the pipe segment a relatively high consequence of failure. This was tempered, however, by its diameter and location within a major arterial street. The 66-inch diameter and location on 35th Avenue both carry a moderate consequence of failure.

Overall, the Deer Valley South Pipeline was assigned a moderate inspection priority. This pipeline was scheduled for inspection first, however, because it was scheduled to be out of service coincident with the shutdown of the Deer Valley Water Treatment Plant. The practicalities of system operation usually take precedent over even the most developed engineering plans.

Predicting and Planning for Shutdown Impacts

Hydraulic modeling was used to identify major system impacts associated with taking each of the 42 large diameter PCCP pipelines out of service. The primary purpose of hydraulic modeling was to identify major operating pressure impacts. Hydraulic modeling provided the additional benefit of highlighting overall system weaknesses and vulnerabilities by identifying critical sections of pipeline which would affect large sections of Phoenix if a rupture occurred.

The City uses a hydraulic water system model run with MWH Soft's InfoWater software. This model is a skeletonized version of the actual water system, incorporating about 1,800 miles of piping primarily 12-inch diameter and larger, six water treatment plants, and numerous pump stations, storage tanks, control valves and pressure reducing valves. Demand and production, from both wells and water treatment plants, are key inputs to the City's hydraulic model.

Water System Demand. Like most municipalities and water agencies, the City sought to limit the impact of pipeline shutdowns by restricting inspections to lower-demand winter months. The demand used for the hydraulic model was the City's forecasted 2010 average day demand of 350 million gallons per day (mgd), with a factor applied to account for seasonal fluctuations in demand. Figure 4 displays seasonal demand fluctuations calculated based on customer usage data for 1994 through 2004. For inspections in October through March a demand factor of 1.0 was applied to the forecasted 2010 demand.

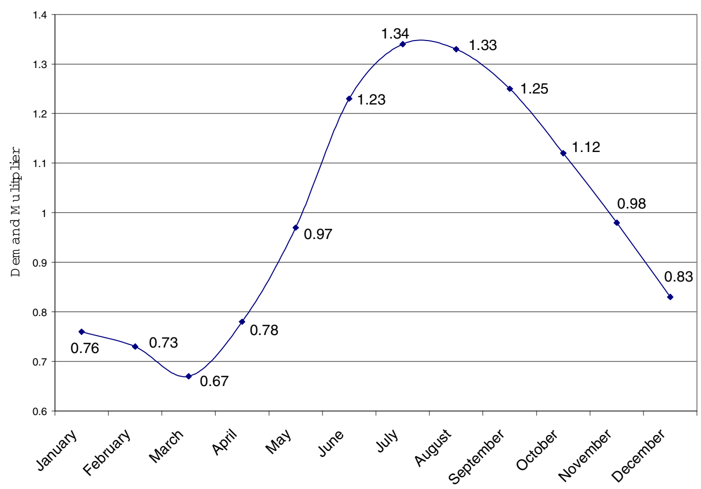

Figure 4. Seasonal Demand Fluctuations

Water System Production. For conservatism and because it is a small part of the City's total water production capacity, well production was discounted. The hydraulic model was run with all plants at typical capacities, for a maximum production of about 570 mgd. The model was also run at lower productions to simulate water treatment plants out of service for planned maintenance and capital improvements.

Evaluation of System Impacts. The hydraulic model was run with all facilities in service to establish a system baseline. Each PCCP pipeline was then systematically taken out of service in the model to identify pressure impacts. To identify which inspections could occur coincident with water treatment plant shutdowns, the model was then run with each water treatment plant and each PCCP pipeline out of service. Although intensive, this process makes full use of the City's hydraulic model and maximizes the quality and quantity of information obtained.

Where the hydraulic model runs highlighted critical pipelines which could not be taken out of service without unacceptable service interruptions, operational modifications and capital improvements were identified to restore service to affected customers.

Modeling Results for the Deer Valley South Pipeline. Hydraulic modeling predicted minimal pressure impacts as a result of taking this pipeline out of service.

Inspection Scheduling

To the greatest extent possible, each PCCP pipeline was scheduled for inspection according to its priority identified by the risk assessment. Where capital improvements were required to take the pipeline out of service, or scheduled water treatment plant shutdowns precluded following the established priority, the inspection was scheduled as soon as possible after capital improvements could be completed or the water treatment plant was back in service.

One month was allowed for each inspection. Activities to be completed in this month include:

- Isolation from the water system
- Dewatering
- Inspection
- Refilling
- Disinfection

An inspection program was developed which seeks to assess the City's entire PCCP system before 2014.

Shutdown Planning

The level of shutdown planning determines the success or failure of the inspection. Because of the short one-month duration of each shutdown, virtually no time is available for reworking the shutdown plan while it is being executed. Development of a sound shutdown plan is critical.

The purpose of shutdown planning is to identify and plan for every factor that could prevent the shutdown, inspection, refilling or disinfection from succeeding. Shutdown plan components must include a plan for exercising and repairing all valves needed to isolate the pipeline from the water system, a detailed public information plan, and a detailed plan for how to dewater, inspect, refill, and disinfect the pipeline.

Valve Management and Shutdown Planning. The purpose of this activity is to ensure all valves are operable before the shutdown begins. This task must be executed far enough in advance of the shutdown to allow time for valve repair or replacement. Valve quantities vary greatly between pipelines. The City expects to exercise at least two 42-inch diameter or larger in-line valves per shutdown, and approximately four smaller diameter side outlet valves per mile of PCCP shutdown.

Public Information. The vast majority of the City's PCCP pipelines are located in major arterial streets. Associated manholes and valve boxes are usually in traffic lanes or sidewalks. Accessing these valves and manholes requires substantial lane closures and other traffic control, and occasionally interferes with business access. The development and execution of a detailed public information plan well in advance of the shutdown is critical to maintaining positive public perception and support of the City's pipeline inspection programs. The public information program communicates the importance of the program to the community and helps nearby businesses and individuals understand how they will be affected.

Shutdown Plan. The shutdown plan is a detailed document which identifies each activity and responsible party required to isolate, dewater, inspect, refill, and disinfect the pipeline. The plan also identifies each valve used to isolate the pipeline and when it will be opened or closed. Specific equipment required to dewater, refill and disinfect the pipeline is specified in the plan. The party responsible for supplying the equipment is also identified.

Conclusion

A targeted and systematic approach to inspecting a large PCCP water system starts with applying risk assessment principles to prioritize the entire PCCP water system for inspection. The overall inspection program should consider the risk-based prioritization as well as:

- System pressure impacts identified using hydraulic modeling
- The time required to dewater, inspect, and recommission a pipeline
- Other previously scheduled capital improvement and maintenance projects

Successful execution of a large-scale, aggressive inspection program hinges on planning. At a minimum, shutdown plan components must include a plan for exercising and repairing all valves needed to isolate the pipeline from the water system, a detailed public information plan, and a detailed plan for how to dewater, inspect, refill, and disinfect the pipeline.

References

Bell, G.E.C. and Romer, A.E., "Solutions to External Corrosion on Buried Water Mains," Advances in Pipeline Engineering and Construction, American Society of Civil Engineers, San Diego, CA, July 2001.

Benedict, R.L., "Analysis of a Corrosion Failure on a Prestressed Concrete Cylinder Pipe," Paper 564, Corrosion 99, NACE, 1999.

Lewis, R.O., "Quality and Performance of Prestressing Wire for Prestressed Concrete Cylinder Pipe," Lewis Engineering and Consulting, 2002.

Price, R.E., Lewis, R.A., Erlin, B., "Effects of the Environment on the Durability of Prestressed Concrete Cylinder Pipe," Pipelines in the Constructed Environment, American Society of Civil Engineers, Pipeline Division, 1998.

INCREASE YOUR DESIGN "BOTTOM LINE" WITH TRENCHLESS SOLUTIONS

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Abstract

In today's competitive engineering design environment, most engineering firms engaged in pipeline design incorporate "labor-saving" technologies into their design process as a business strategy to improve the "bottom line". Although these tools are aimed at raising productivity, increasing profit, and streamlining design methods, there's often little consideration as to what is designed rather than simply the design process, as a means of increasing profitability. Simply put, when considering water and storm and sanitary sewer rehabilitation or replacement, it is time to rethink the traditional engineering design business strategy of designing the "same old thing, but faster".

Integrating trenchless technologies into the engineering design process provides the opportunity to change the "what is designed" element of the design profitability equation. In this paper, the author presents the findings of his review of more than 50 US trenchless projects designed and bid in 2006 and then compares them to similar "dig and replace" designs. The author concludes that most "trenchless" designs were over-designed and frequently included unnecessary "dig and replace" details. When comparing "trenchless" designs to comparable "dig and replace" projects, the data illustrates that efficiently designing a trenchless project can easily result in significant savings in design time, a decrease in the potential for errors, and sizable increases in design profit (especially in Design-Build projects). The author also provides specific essential trenchless information to help design trenchless efficiently and to level the trenchless bid or evaluated proposal "playing field."

Discussion:

Many engineering design firms have sought to increase design productivity and profits by utilizing traditional designs except with hopefully lower production costs. However, when considering the design decision between replacing a customer's deteriorated underground infrastructure (water lines and storm and sanitary sewers) or rehabilitating it with trenchless methods, often little thought is given to designing trenchless as a business strategy for increasing the firm's design "bottom line." There are many possible reasons for this phenomenon; however, the purpose of this paper is to prompt a business case reconsidering designing "dig and replace" or trenchless underground infrastructure project solutions. It is time to reconsider "what we design" as a means of increasing design profitability and efficiency, rather than just "how we design." This paper presents a business case for designing trenchless rehabilitation rather than "dig and replace" solutions and illustrates that trenchless design can increase design profits.

An Introduction to Trenchless Rehabilitation Solutions:

Trenchless rehabilitation solutions have become mainstream to underground infrastructure rehabilitation since their introduction in the 1970s. They share the obvious characteristic of "little or no digging" to rehabilitate existing lines and the same general processes of first televising and cleaning the existing pipe and then examining existing conditions for significant flaws that would exceed design criteria and might warrant a surgical "point repair". A "new pipe system" is then designed and manufactured for the specific application and, after installation, service reconnects are installed, (usually robotically) from the inside of the restored pipe. The completed installation is again televised to ensure installation quality and to provide a benchmark for the owner's asset management program.

As an introduction to specific trenchless solutions, there are four families of trenchless technologies: Cured-in-Place Pipe (CIPP), Formed in Place Pipe (FIPP), Pipe Upsizing (Pipebursting) and Sliplining considered appropriate for pipe renewal and replacement applications. In addition, horizontal directional drilling and microtunneling are trenchless solutions installing new pipe in new locations; however, as the replacement or renewal of existing pipe is the more common design decision and the focus of this paper, these technologies are not detailed.

CIPP is the most widely practiced trenchless technology and consists of felt tubing that hosts a resin system and a curing process (usually steam or hot water) to activate and "cure" the resin system. CIPP is a fully structural solution with a design life often exceeding 100 years, based upon American Society of Testing Materials (ASTM) (ASTM D790, 2006) extrapolation of the long-term buckling pressures beyond the 10,000 hour test period and comparisons with long-term in-ground performance. The CIPP process rehabilitates pipes from ½" to 108" in diameter in seamless/jointless lengths up to 3000 feet. Millions of feet of CIPP have been placed in the ground in the last 35 years.

FIPP material is approved for potable water use and is ideal for pressure applications, such as water supply systems from 2" to 48" in diameter and where exterior structural loading is not an issue.

Pipebursting cuts or bursts an existing pipe from inside and then pulls a welded pipe behind the cutter/burster into the enlarged hole. Pipebursting can increase the original flow capacity. Depending upon the insitu soil or rock conditions and the depth and diameter, increases in diameter from 0%-25% are commonplace; 25%-50% increases often challenging, and 50%-125% diameter increases are considered experimental.

Finally, in the sliplining process, a smaller pipe is pulled or pushed inside the existing, larger pipe. Both water and sewer pipes from 8" to 96" in diameter can be rehabilitated using the sliplining technique.

Trenchless technological improvements have been rapid and significant, especially within the last five years. The industry has seen impressive performance enhancements. For example, accelerated performance and strength testing for many trenchless systems show that design life exceeds 100 years…and still counting. CIPP inversion with air and curing with steam has addressed water/head pressure issues on large diameter and long, near-vertical applications and reduced cure times to a few hours, with significant reductions in water and energy consumption and elimination of environmental impact from the curing process.

Internal reinstatement of laterals and service connections on sewers is commonplace; however, recent introduction of robotic reinstatement of water line service connection from within the pipe, eliminated the need for digging to reinstate water connections. Technological improvements have improved performance and consistency by adopting ISO 9000 standards for both products and installation. Additionally, composite materials have been introduced to increase strength and optimize performance (with moduli of elasticity more than six times greater than previously) while reducing thicknesses and material requirements. As a result of rapid technological and operational advances, when trenchless bid or commercial pricing guide costs are compared to "dig and replace" prices or "trenchless versus 'dig and replace'" bid options, 30%-60% project cost savings can be realized by using trenchless methods (Means, 2006). Additionally, 2000-2500 linear feet rehabilitated per week production rates are attainable. Further, trenchless construction virtually avoids the impact of weather, varying site conditions, right-of way access issues inherent to "dig and replace" construction, thus greatly improving trenchless' relative time and cost savings.

The design business and trenchless technologies:

Civil design management is neither a frequently researched nor published topic. Although design services are typically acquired by best-value, negotiated procurements, design productivity and pricing information remain well-guarded secrets within most design firms. The business case for designing trenchless solutions has been discussed even less.

There are several reasons the design industry has not wholly embraced trenchless technology as a design solution to pipeline rehabilitation and repair problems. The first challenge is that trenchless technologies have changed rapidly and continue to change dramatically. The rate of change in trenchless technologies, including new and sophisticated materials, processes, procedures and prices, has increased sharply in the last five years. For example, the practical ability to robotically reconnect service connections from within a water line didn't exist outside of the laboratory as late as a year ago. Also, trenchless rehabilitation solutions are generally less than 40 years old and few engineering schools expose their students to trenchless technologies or teach them how to design trenchless solutions. As a result, relatively few design engineers know how to efficiently design trenchless solutions, and few business managers understand the financial and production benefits of designing trenchless solutions.

Another significant obstacle to designing trenchless is the paradox that design fees are typically based upon a percentage of the total project cost and, as trenchless solutions can reduce the project cost dramatically, the potential reduction in design revenue and profit would appear to be significant.

Determining the business case for designing underground infrastructure renewal/replacement:

The question then becomes, how can design firms increase design productivity and profit by designing trenchless and without reducing profits. To answer this question, in 2006, the author randomly selected and reviewed bid documents (completed plans and specifications) for 52 trenchless projects from across the United States. The project costs ranged from approximately $33,000 to $3.2 million. The projects were designed by a wide range of engineering firms; large and small civil and general engineering firms, and firms with national or area presence, and the customers ranged from very large to small municipalities.

The first conclusion was that most trenchless designs contain too much information. Of those 52 projects, only three (approximately six percent) designs provided sufficient information (i.e. enough information to establish trenchless design parameters and production rates) without adding unnecessary information to drawings and specifications. From a review of the bid tabs, these three optimally designed projects realized bidding competition and very competitive prices.

To compare the visible design effort and production costs for designing trenchless instead of "dig and replace" solutions, the three "efficient" trenchless designs were compared to eight "dig and replace" designs that provided only necessary information (i.e. rights of way and utility information, sod, pavement, curbs, and sidewalk patching details, environmental control measures, and valve, fitting, and pipe bedding details…to mention a few). These "dig and replace" designs represented roughly the same cross-section of scope, locale, customers and large and small design firms.

The results of the comparison between efficient "dig and replace" and trenchless designs revealed that "dig and replace" designs required an average of 40% more design effort (as measured by numbers of job-specific drawing details beyond the site-specific scope description and pages of applicable and job-specific technical provisions) than comparable trenchless projects. In fact, in the optimized trenchless designs, there was typically little additional drawing and surveying effort beyond the site-specific stage of the "dig and replace" project. Simply stated, trenchless design is essentially complete after the scope of work is detailed…while the "dig and replace" design is only 60% completed.

Of course, design effort and productivity is much more complicated than what these findings imply. However, the fact remains that when "dig and replace" is the only solution considered for rehabilitating existing lines, a design firm loses an opportunity to determine its own profit advantage gained by designing trenchless.

As an example, of the profit calculation, a typically sized "dig and replace" construction project from this study cost roughly $300,000 to construct and covered the same length of pipe as the trenchless project which cost approximately $200K. Assuming a 10% design fee percentage, the "dig and replace" project yielded a $30K design fee, while the trenchless design fee was $20,000. Assuming a similar early design phase for both projects and straight-line profit-earning and design cost ratios, trenchless design's ability to be complete at 60% of the "dig and replace" design phase, would roughly cost 60% of $30,000 or $18,000. The additional profit beyond the straight-line profit-earning curve would be $20,000 minus $18,000 to design; an additional $2,000 (or 16%) above the design profit percentage.

However, 16% additional profit is probably conservative. The bulk of the trenchless design (and design cost) is done in the "field work" design stage, by survey, field verification and computer-aided drafting (CAD) or Geospatial Information System (GIS) personnel. The extensive use of lower ranking design members (involved heavily in the early (i.e. site-characterization phase) of the project) would indicate that the actual design costs would be much lower at the beginning than at the end of the project. As a result, it is very reasonable to assume that design profit earnings would be significantly higher for the first half of a project than for the latter half and, as a result, a 16% increase in design profit is conservative.

But the real lessons about production efficiency when designing trenchless versus "dig and replace" solutions are found in the other 49 projects that were overdesigned. The common thread running through these overdesigned projects was that they contained information needed for a "dig and replace" design, although "dig and replace" was rarely an optional bid item. As a result, "dig and replace" details such as connection and bedding details, pipeline profiles, manhole cover details, valves, etc. were incorporated in the drawings and specifications. The reasonable conclusion is that the designer did not realize that trenchless solutions require just the site, loading and physical information that a "dig and replace" design requires. At that point, the trenchless design is nearly complete…while the "dig and replace" design is essentially just starting.

Adding additional information to cover every conceivable trenchless solution was a second trait. The additional details might have widened the bidding field, but at what cost? In the trenchless industry, introducing processes relying heavily on key features that work well in the laboratory but are highly improbable in the field is considered fraught with the potential for problems. From an engineering perspective, the more complicated a process, the more likely problems can occur. In these cases, design decisions boil down to choosing an appropriate family of trenchless solutions that minimize life cycle costs for the client, design risk and design costs. Typically, CIPP and

FIPP trenchless solutions require the fewest amounts of design details and time. Arguably, they also exhibit the fewest problems over their design life.

How do I design trenchless?

In a nutshell, essential trenchless design information allows the trenchless contractor to determine thickness design parameters, as specified by ASTM, and installation production rates, which in turn, largely determines price (ASTM F1216, 2006). The minimum design information therefore, is site location, manhole placement, pipe diameters, lengths of run and sharp changes in direction, depth of lines, laterals (if known), groundwater (if an issue), access to the pipe, installation staging areas, flow data (for bypassing), unusual loading and subsurface conditions, and any special features. Again, this information, in essence, is the starting point for a "dig and replace" design.

The Collateral Benefits of Designing Trenchless:

There are other collateral benefits that are derived from designing trenchless aside from increased design productivity and improved "bottom line." The first is a decrease in risk…both to the owner and the designer. First, as trenchless technologies are relatively immune to the effects of polluted ground water and soils that surround the pipes that are being rehabilitated, there is seldom a requirement to remediate as the polluted area is not exposed or posing an environmental hazard. Similarly (especially in the case of asbestos cement pipe, which poses a removal and disposal hazard when disturbed) "hazardous" pipelines can be rehabilitated and effectively encapsulated by trenchless technologies. This is especially true for CIPP, which does not necessarily rely upon pulling the replacement pipe into the host pipe and dislodging hazardous materials.

For the designer, as has been shown, there are fewer items to design and significantly reduced potential for errors and omissions. Additionally, trenchless technology firms provide design safeguard in that they recalculate required thicknesses based upon the actual conditions observed from the video tapes and existing conditions. Further, ISO 9000 quality certifications for trenchless products and installation are available.

Finally, trenchless solutions also possess sustainable qualities that "dig and replace" solutions do not. One measure of sustainable design, the US Green Building Council's "Leadership in Energy and Environmental Design (LEED)" program currently focuses on buildings; however, their focus upon minimizing manufacture, transport and installation energy costs and the environmental consequences of using products apply to underground infrastructure as well as buildings (USGBC, 2007). Trenchless solutions squarely address more than a dozen "site development" and "reuse of existing materials" categories in various LEED evaluation systems and are often awarded "innovation" credits for including technology that improves upon "dig and replace." As a result, contrary to the perception that LEED solutions increase project cost, trenchless technologies can actually improve LEED ratings…at a cost savings.

Additionally, trenchless technologies fare quite well when evaluated using "Environmental Building News" (www.buildingGreen.com-EBN, 2006) "Five 'Green' Criteria" (i.e. Products that 1. are made from salvaged, recycled or agricultural waste; 2. conserve natural resources; 3. avoids toxins or emissions; 4. save energy or water and 5. contribute to a safe, healthy, built environment). Considering trenchless' ability to minimize pollution while reducing water and energy waste, and its' incredibly long design life, trenchless technologies certainly exhibit "green/sustainable" characteristics that "dig and replace" solutions could never hope to achieve.

Design-Build and designing trenchless:

Many inherent trenchless advantages are leveraged further when applied to design-build project delivery because trenchless' speed of installation and minimal impact upon the site (i.e. few if any excavations) capitalize on design-build's fast tracking capabilities. Incorporating and collaborating with a trenchless solutions contractor early in the design-build process not only ensures that the latest technologies and benefits are realized, but also that the trenchless contractor can provide the trenchless design for approval. Additionally (as trenchless rehabilitation is not on the construction critical path) the designer no longer has to scramble to stay ahead of construction and can design practically at will. When a trenchless contractor is a member of the design-build team, the design engineer also benefits from trenchless' ability to lock-in price almost as soon as scope is determined. The entire design-build team also benefits from reduced design and construction time and cost, reduced weather and environmental risks and reduced impacts upon schedule. Finally, a certified trenchless contractor can assure ISO 9000 products as well as ISO 9000 installation quality.

Conclusion:

Any design firm can gain a number of benefits for themselves as well as their client, by reconsidering the "dig and replace" approach to rehabilitating existing water and sanitary and storm sewers. Design firms can increase their own design profitability while also serving their client with lower life cycle costs, longer design life, less disruption, faster completion and less vulnerability to unforeseen site conditions such as unknown utilities, polluted soils, and differing site conditions. Trenchless solutions also provide the collateral benefits of minimizing design and performance risks while providing a sustainable solution that preserves the environment and energy resources.

References:

American Society of Testing Materials (ASTM). (2006). *D790-06: Standard Test Methods for Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials*, ASTM International, West Conshohocken, PA

Means, R.S. (2005). *Facility Construction Cost Data, 2006, 21st Annual Edition* Construction Publishers and Consultants, Kingston, MA.

American Society of Testing Materials (ASTM). (2006). *F1216-06: Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin-Impregnated Tube*, ASTM International, West Conshohocken, PA.

United States Green Building Council. (2007). *Leadership in Energy and Environmental Design*, www.usgbc.org .

Environmental Building News. (2006). http://www.buildingGreen.com/EBN/checklist.cfm

SEWER HYDRAULIC DESIGN CRITERIA

Richard E. Nelson[1]

ABSTRACT

Many sewer agencies and consulting engineers still use older "tried and true" formulas, graphs, and guidelines for sewer hydraulic design. With increasing attention on system performance and the need to demonstrate capacity assurance, it is important to re-examine sewer hydraulic design criteria. The relevancy of standards such as 10 States Standards and individual state sewer design criteria, and sewer agency in-house design criteria should be reviewed relative to cost/benefit analysis considering infiltration and inflow (I/I) rates.

The wastewater industry has been performing detailed flow studies for over 30 years. What has been learned can be applied for creating better sanitary sewer hydraulic design criteria to improve system performance by reducing the future occurrence of SSOs. In this paper, various design criteria are reviewed relative to the impact on sizing, cost, and system I/I.

Historical Sewer Sizing Considerations

Simply put, sewers are sized to convey the peak design flow rate from all contributing sources including an allowance for infiltration and inflow (I/I). Design of new sewer systems is based on state and local regulations that provide for the minimum design standards, acceptable methods of estimating peak flows, and minimum velocities to ensure that materials are scoured and do not settle and permanently build up during periods of low flow.

Typically, estimates of peak flow are calculated using accepted equations that multiply the average daily flow (ADF) by a peaking factor to determine the peak design rate. These peaking factors are an attempt to empirically account for I/I since dry weather peaking factors are on the order of 1.2 to 2.0 while design flow peaking factors are on the order of 2.5 to 6.0 or greater.

It is clear from reference documents and experience, that I/I is a critical consideration for design in systems that experience system flow responses to rainfall. ASCE Manual No. 60 states that, "Sanitary sewer design capacity must include an allowance for extraneous water components which inevitably become a part of the total flow" (ASCE). The manual provides examples of design criteria and design curves from a number of agencies however no guidance is provided relative to what the flow curves mean in terms of risk or probability of exceedance or what or how to make appropriate adjustments based on level of service desired. Example design flow equations from the ASCE manual are presented in Table 1. The basis of design flow varies from a fixed unit based method to a population based method that result in curves shown in Figure 1. One of the curves in Figure 1 dates back to 1918.

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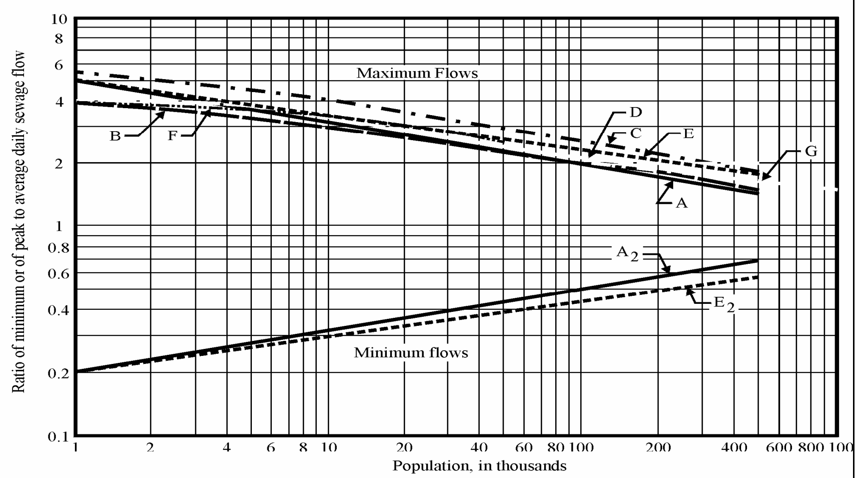
Table 1
Typical Design Flows

City	Year and Source Data	Average Sewage Flow (gpd/cap)	Sewer Design Basis (gpd/cap)	Remarks
Boston, MA	1980	264	100*	Includes infiltration. Multiply by 3 when sewer is flowing full.
Des Moines, IA	1980	100	100 x factor*	*Factor = $\frac{18 + \sqrt{P}}{4 + \sqrt{P}}$ P = population in thousands
Kansas City, MO	1980	160	0.01 cfs/acre 0.02 cfs/acre	For interceptors For laterals and sub-mains
Lincoln, NE	1964	60		For lateral sewers maximum flow by formula: Peak flow = 5 X avg flow / (pop in thousands)^0.2
Milwaukee, WI	1980	225	275*	*Plus additional factors for inflow/infiltration

Source: Gravity Sanitary Sewer Design and Construction, ASCE Manual of Practice, No. 60

Figure 1 - ASCE Design Curves from Manual No. 60

Figure 2: Ratio of extreme flows to average daily flow compiled from various sources.



Curve A source: Babbitt, H. E., "Sewerage and Sewage Treatment." 7th Ed., John Wiley & Sons, Inc., New York (1953).
Curve A_2 source: Babbitt, H. E., and Baumann, E. R., "Sewerage and Sewerage Treatment." 8th Ed., John Wiley & Sons, Inc., New York (1958).
Curve B source: Harman, W. G., "Forecasting Sewage at Toledo under Dry-Weather Conditions." *Eng. News-Rec*. **80**, 1233 (1918).
Curve C source: Youngstown, Ohio, report.
Curve D source: Maryland State Department of Health curve prepared in 1914. In "Handbook Of Applied Hydraulics." 2nd Ed., McGraw-Hill Book Co., New York (1952).
Curve E source: Gifft, H. M., "Estimating Variations in Domestic Sewage Flows." *Waterworks And Sewerage*, **92**, 175 (1945).
Curve F source: "Manual of Military Construction." Corps of Engineers, United States Army, Washington, D. C.
Curve G source: Fair, G. M., and Geyer, J. C., "Water Supply and Waste-Water Disposal." 1st Ed., John Wiley & Sons, Inc., New York (1954).
Curves A_2, B, and G were constructed as follows:

Curve A_2, $\frac{5}{P^{0.167}}$

Curve B, $\frac{14}{4 + \sqrt{P}} + 1$

Curve G, $\frac{18 + \sqrt{P}}{4 + \sqrt{P}}$

in which P equals population in thousands.

Source: Gravity Sanitary Sewer Design, ASCE No. 60

For systems with significant I/I, the normal wastewater flow may be a relatively small fraction of the total peak flow used for design. Continuous design curves represent an improvement over unit rate flow estimates since continuous curves account for system dynamics and the changing peak to average day flow ratios that occur as flow passes through the system. The flow curves are primarily empirical in nature and have provided valuable design guidance for many years. Nevertheless, in today's changing regulatory environment and the need to optimize designs, design flow curves should relate flows to a risk of exceedance and should be correlated to monitored data whenever possible. The curves as presented in Table 1 and Figure 1 do not express the design flows in terms of risk (level of service, in other words, protection against sewer backups and sewer overflows).

Flow Components

Flow monitoring data from existing systems can be used to define flow components and for generating design flow curves. For new areas, either monitored data from areas with similar characteristics or estimates based on experience can be used to select flow parameters from which to generate design flow curves. Suggested flow components to consider for generating a design curve include:

1. wastewater production (WWP) or base flow,
2. infiltration, and
3. inflow or rainfall dependent I/I (RDII) for a selected storm event.

Each of these flow components in defined in the following paragraphs.

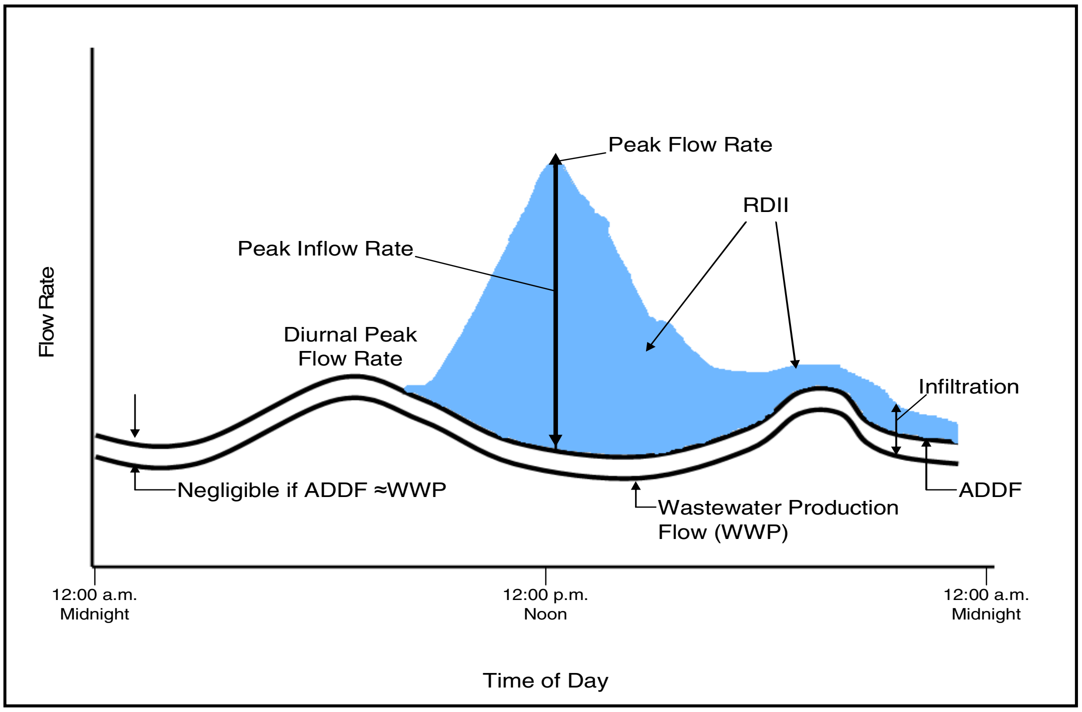
Wastewater Production (base flow). Wastewater production (WWP) flow is the flow generated in a sewer system exclusive of infiltration and inflow. WWP is *not* the average daily flow (ADF). The ADF flow parameter includes infiltration and inflow volumes and is based on the annual volume of flow divided by the number of days in a year. Estimates of WWP should be made based on an analysis of land use through the planning period and on monitored data. Evaluation of unit flow rates, such as gallons per capita-day (gcd) can provide some insight into the validity of the estimate. WWP can also be estimated based on water consumption records, land use, and/or population estimates and selected unit flow rates.

Infiltration. Infiltration is defined as groundwater related flow entering the sewer system through defective pipes, pipe joints, and the lower sections of manholes. Estimates of infiltration can be based on monitored data during high groundwater/no rainfall periods or on experience. Total infiltration is typically estimated by taking the difference between monitored flows during dry weather/high groundwater conditions and the estimated WWP.

Inflow. Inflow or rainfall dependent I/I (RDII) is defined as rainfall related water entering the sewer system through defects or undesirable direct or indirect connections. Inflow is highly variable and is dependent, in part, on antecedent conditions as well as the depth and intensity of rainfall. Inflow can be the largest flow contributor under peak flow conditions for systems experiencing high rates of infiltration and inflow (I/I).

The total peak design flow should include wastewater production, infiltration, and inflow. These flow components are shown in graphical form on Figure 2. Because inflow is dependent on rainfall, development of design flows requires an evaluation of risk and selection of a level of service.

Figure 2 - Design Flow Components



Development of Design Flow Curves

For existing sewered areas, flow components are best determined from flow and rainfall monitoring data. For future areas to be served, flow components can be estimated based on land use, flow-monitoring data in similar types of areas and judgment. Flow components may be determined as described in the following paragraphs.

Wastewater Production. Wastewater production (WWP) can be based on water consumption records, standard tables showing projected wastewater production for residential, commercial and industrial use, and/or flow monitoring data during dry weather/low groundwater conditions. For estimating WWP from flow monitoring data, a minimum of seven days during the most relevant low flow period should provide adequate flow data. If significant flow variations exist between weekend and weekday periods, the critical period should be selected. The diurnal peaking factor should be determined or estimated based on monitored data or from data from similar areas. The diurnal peaking factor is the peak hourly rate divided by the average day rate during dry weather.

Infiltration. Infiltration should be measured during nighttime flows following storm events when infiltration is at its highest and flow includes a negligible amount of inflow. Experience has shown that the minimum three hour nighttime flow is effective for measuring infiltration. Nighttime is particularly effective for measuring infiltration because the effects of other flow components are at a minimum and better meter resolution for flow differences can be obtained when flow depth is at a minimum. Infiltration can be estimated using flow data about 48 to 72 hours after storm events by averaging the minimum 3-hour nighttime flow and subtracting the nighttime flow minimum which occurs during dry weather/low groundwater (ADDF) periods. If the measured ADDF includes significant qualities of infiltration, then the estimated amount of infiltration occurring during monitored dry weather/ low groundwater conditions should be added to the measured infiltration to obtain the total infiltration. For growth areas, infiltration rates should be estimated based on flow monitoring data from similar areas and judgment.

Inflow. Inflow is a function of many variables. There are a number of methods to estimate and project inflow that are described in The Water Environment Research Foundation (WERF) research project conducted in 1999 that grouped infiltration and inflow hydrograph generation into eight categories (WERF):

- Constant unit rate methods
- Percentage of rainfall volume (R-value) methods
- Percentage of stream-flow methods
- Synthetic unit hydrographs methods
- Probabilistic methods
- Predictive equations based on rainfall/flow regression
- Predictive equations based on synthetic stream flow and basin character, and
- Rainfall-derived infiltration and inflow as a component of hydraulic software

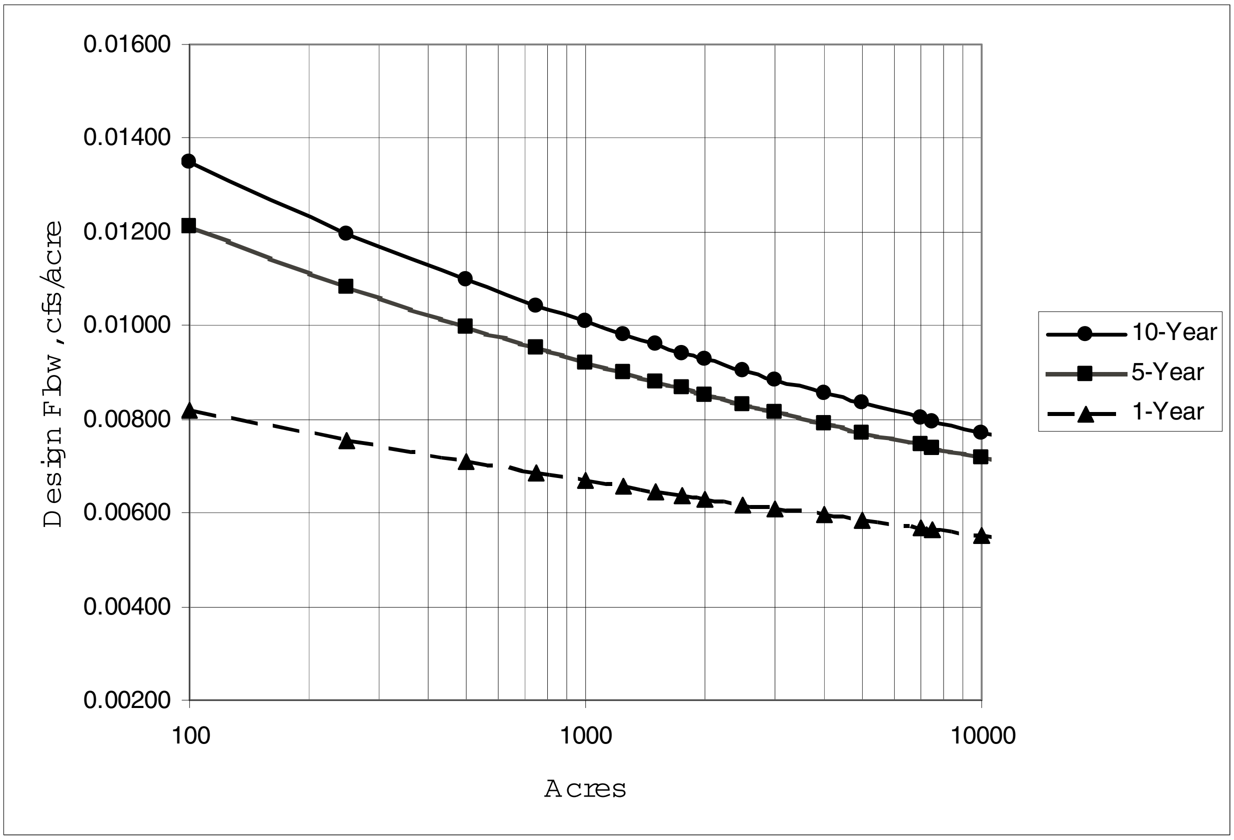
In practice, the author has used a simplified method, called the *inflow coefficient method* that is a combination of a predictive equation and synthetic hydrograph and can be used to estimate peak inflow for selected storm events. The *inflow coefficient method* equation is as follows:

$$Q=K_P iA \qquad [1]$$

Where: Q = Inflow rate, cfs
i = average rainfall rate for the time of concentration to the metering point, in/hr.
A = sewered area, acres.
K_P= inflow coefficient for peak flow

Various surrogate variables can be substituted for the area as appropriate or desired. For example, if population data is used, population densities can be used to calculate the surrogate for the area in inflow calculations. Figure 3 shows a family of flow curves generated using the inflow coefficient method for the 1-, 5- and 10-year recurrence interval. Sizing requirements can be evaluated for alternative levels of service using this type of curve.

Figure 3 – Design Flow Curve Generated Using Inflow Coefficient Method of Analysis



Other Design Criteria

In addition to design flow, other key design considerations that will determine the size of pipe for design include:

- Minimum and maximum velocities
- Ratio of flow depth to diameter (d/D) used to size sewers
- Friction factor for sizing sewers

Minimum and Maximum Velocities

It is generally accepted that minimum design velocity should not be less that 2 feet per second or generally greater that 10 feet per second (fps) at peak flow. In recent years however, it has been demonstrated that a single minimum velocity, which is unrelated to the characteristics of the wastewater, does not realistically represent the ability of sewer flows to transport sediment. In particular, larger sewers may require a substantially higher flow velocity in order to be able to transport the same concentration of sediment as small sewers. Report 141, Design of Sewers to Control Sediment Problems, by CIRIA presents research and design procedures to consider depending on the type of flow regime and wastewater characteristics (CIRIA). For what is termed "cohesive sediment" erosion (Criteria III) by the report, the following minimum velocities are required to achieve the appropriate bed shear stress. Generally, this type of sediment is found in separate sewers, storm sewers and combined sewers.

Table 2 – Minimum Velocities Recommended for Criteria III Sediment

Pipe Diameter, in	Minimum Full Pipe Velocity
8	2.4
12	2.5
15	2.5
18	2.6
24	2.7
30	2.8
36	2.9
42	2.9
48	3.0
60	3.0
84	3.1
96	3.2

During design, careful consideration should be given to achieve required scour velocities that may exceed the typical 2 fps "rule".

Ratio of flow depth to diameter (d/D)

Another important design criterion is the design flow depth to diameter (d/D) for the design peak flow. Table 3 presents d/D sewer sizing criteria from several sources. The data show that there is a mix of allowable d/D being used in practice. Designing pipe size with a d/D equal to 1.0 does not provide any margin of safety for actual flow deviations from projected, line debris which may accumulate, or actual construction slope differing from planned. The selected d/D however, must consider the overall impact of all design variables including the selected design flow recurrence interval, friction factors, and the uncertainty in flow projections. For example, a sewer line sized with a d/D of 0.75 and a 1 year recurrence interval could be approximately the same size as a line sized with a d/D of 1.00 and a 10-year recurrence interval.

Table 3
Sewer Sizing Criteria

Description	d/D >= 18 inches	d/D < 18 inches
Lincoln, NE	1.00	1.00
Missouri, State of	1.00	1.00
Ten State Standards	1.00	1.00
Kansas, State of	0.75	0.67
Clay Pipe Manual	0.75	0.50
ASCE Manual No. 60	0.75	0.50

Friction Factors

Pipe friction factors are well documented in many technical references. The hydraulic design friction factors should also consider the following:

- Pipe material
- Potential for pipe wall deposition buildup especially in areas prone to grease buildup or deposition buildup
- Pipe connections
- Pipe transitions and special structures

Generally for small diameter sewers (less than 18 inches) a pipe Manning's 'n' value of 0.013 – 0.014 is appropriate to use for design. For large sewers, a pipe Manning's 'n' value of 0.011 – 0.013 is generally appropriate to use for design. In addition to the pipe losses, it is important to factor in additional losses at manholes and special structures as well as bends.

Example Impact of Design Criteria on Sewer Sizing

As evidenced by the information presented in this paper, there are a number of key considerations for hydraulically sizing sanitary sewer lines. Some of the variables have greater impact than others but once selected, they determine the size and slope of the sewer to be constructed and consequently impact on costs of the overall improvement. Table 4 presents a hypothetical example of sewer sizing using selected criteria and the following assumptions:

- An area of 1000 acres
- Peak flow as represented in Figure 3
- Slope equal to 0.1%

The least conservative design is Condition 1 and 1 Yr Service. The most conservative design is Condition 3 and 10 Yr Service. The range of pipes required, for the same service area using different design criteria, ranges from 24-inches in diameter to 42-inches in diameter. Based on observation of this table it appears that the most significant factors, from greatest to least is, the d/D selection, the level of service and finally the friction factor. In this example, the pipe size for a 10-year level of service for Condition 1 is the same as the pipe size for the 1-year level of service for Condition 2. Accordingly, all design variables must be understood to properly evaluate the adequacy of improvement size.

Table 4
Example Impact of Design Variables – Pipe Diameter in Inches Required

	Recurrence Interval	***1 Yr Service***	***5 Yr Service***	***10 Yr Service***
Design flow from Figure 3	*Design flow, mgd*	*4.3*	*6.0*	*6.5*
Condition 1	Manning's n = 0.011, d/D = 1.0	24	27	27
Condition 2	Manning's n = 0.013, d/D = 0.75	27	30	30
Condition 3	Manning's n = 0.014, d/D = 0.5	36	42	42

Note: "1 Yr Service" etc. refers to the design flow is not expected to be exceeded, on average, during a 1 year period.

Conclusions

This paper presents considerations for hydraulic design of sewer lines by evaluating peak flow rates with special emphasis on I/I and level of service (risk), design d/D, design velocities, and design friction factors. In summary, the following conclusions are offered:

1. Design flow curves should consider risk (level of service).

2. Rule of thumb design velocities may not be adequate to properly scour sewer systems and reassessing the design velocities is important to prevent future maintenance problems.

3. There are a variety of d/D design values used and selection must be weighed considering all design factors so that neither an overly conservative nor overly risky design results.

4. Pipe friction factors are well documented however all hydraulic impacts should be considered including transitions and special structures when sizing sewer lines.

5. Level-of-service and d/D generally have the most impact on sewer sizing.

References

ASCE, (1982) "Gravity Sanitary Sewer Design and Construction, Manual of Practice No. 60".

CIRIA (1996) Construction Industry Research and Information Association, "Report 141, Design of Sewers to Control Sediment Problems."

WERF, Water Environment Research Foundation, (1999) "Using Flow Prediction Technologies to Control Sanitary Sewer Overflows", Project 97-CTS-8.

VALIDATION OF A DECISION SUPPORT SYSTEM FOR METHOD SELECTION IN UTILITY CONSTRUCTION

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Abstract

The decision of how to accomplish the installation or repair of a buried pipe in an urban environment involves tangible and intangible parameters. This paper outlines the development and implementation of comprehensive, yet straightforward and easy to use interactive software for the evaluation of alternative construction methods that can be employed in the installation or replacement of buried pipes and conduits. The software emphasizes simplicity and practicality, and limits input data to those readily available to utility engineers at the design stage of the project. The program takes into account extensive performance data for 20 new construction methods and 6 in-line replacement methods commonly used in utility projects, and was designed to raise awareness and provide guidance to the utilization of trenchless technology methods across the buried utilities arena. This paper focus on the validation phase of the model development, reporting the results of a comparison of the model's predictions with the out come of five case histories, which cover a wide range of project requirements, soil conditions and environmental constraints.

Background

This project was commissioned by the National Utility Contractors Association (NUCA) Trenchless Technology Sub-Committee and is intended to be a companion to NUCA's Trenchless Construction and Rehabilitation Methods Manual (4th Edition). The program, titled 'Trenchless Assessment Guide' (TAG), was designed as stand-alone software to assist municipal and utility engineers in evaluating the technical feasibility of various traditional open cut, trenchless construction, and inline replacement methods for a specific project. TAG is compatible with Microsoft operating platforms such as Windows XP and Windows 2000. The program relies on an extensive built-in database containing performance data for 26 construction methods commonly used in utility projects.

The objective of this project was to develop and codify an algorithm that accomplish the following: a) perform a sound technical evaluation as a screening measure to eliminate incompatible construction methods; b) evaluate and quantify the overall perceived risk associated with the competing alternatives; and, c) raise awareness and provide guidance to the utilization of trenchless technology methods. A full version of TAG is available at http://www.nuca.com/i4a/store/category.cfm?category_id=4.

Method Database

The relational method databases contain a plethora of information about each construction method. The general method information section includes a detailed description, a color picture and the expected environmental impact and extent of excavation (continuous vs. access/receiving pits only). The databases also contain information about the method's technical capabilities, such as maximum and minimum pipe diameters, drive lengths and allowable depths of cover. The databases also contain pipe compatibility information for ten commonly used pipe materials, soil compatibility information for ten types of geological materials and ground water table limitations.

The relational method database contains information about 26 construction methods, including 18 trenchless methods for placement of a new pipe, six of which are inline replacement methods, and two open cut methods. The construction method database is updatable, customizable and expandable. The user can easily add new methods or pipe materials, and update the capabilities of existing methods as technology develops and new innovations are introduced into the trenchless market. Changes can be made directly from the construction method database forms by inputting new values and pressing the 'Update' button. The database can also be expanded from the Microsoft Access database file. Thus, the software is expected to remain a 'living application' for a prolonged period of time. Figure 1 shows a sample method database form for track type Auger Boring from TAG.

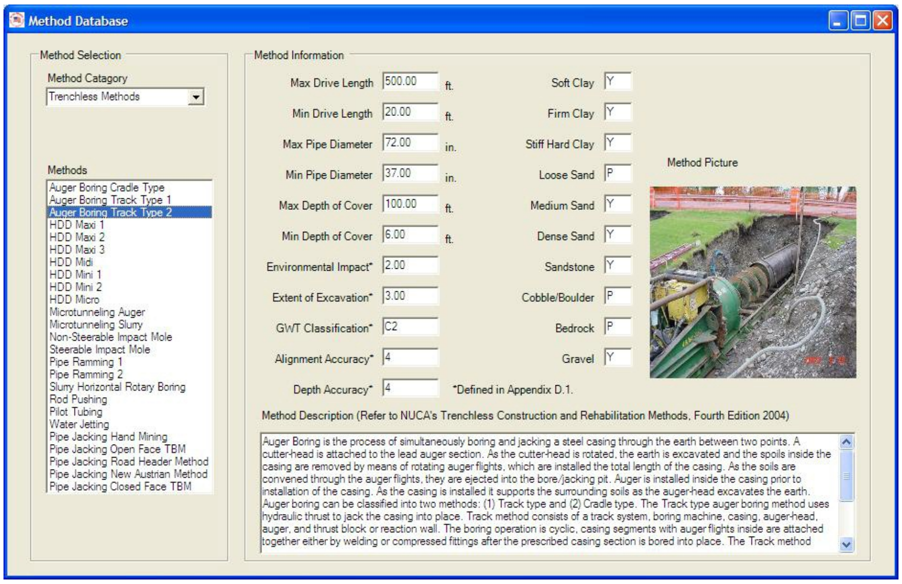


Figure 1. Database form for track type Auger Boring.

Technical Evaluation

The technical evaluation begins by defining the type of problem the user is facing. It is believed that all buried pipe problems can be reduced to either a structural problem or a capacity problem. TAG incorporates a built-in wizard, which is based on a series of interactive questions presented to the user. Based on the user's answers, certain categories of construction methods might be eliminated. Figure 2 shows one of the wizard's forms which contain a set of interactive questions for a capacity problem.

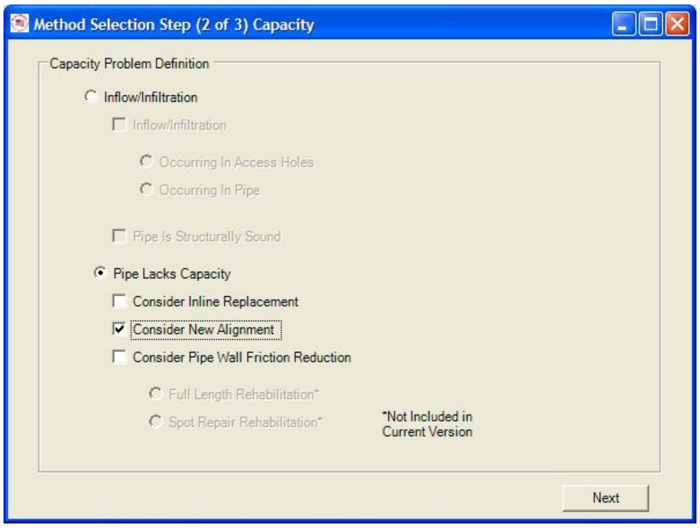


Figure 2. Interactive questions for a capacity problem.

The next step in the technical evaluation is the input of the project specific data. Four categories of information are input during this stage. The first category includes project specific parameters such as drive length, pipe diameter, depth of cover and elevation of the ground water table, shown in Figure 3. Also included in the input are the anticipated degree of accuracy of alignment and profile, which are defined in the TAG user's manual (NUCA, 2006).

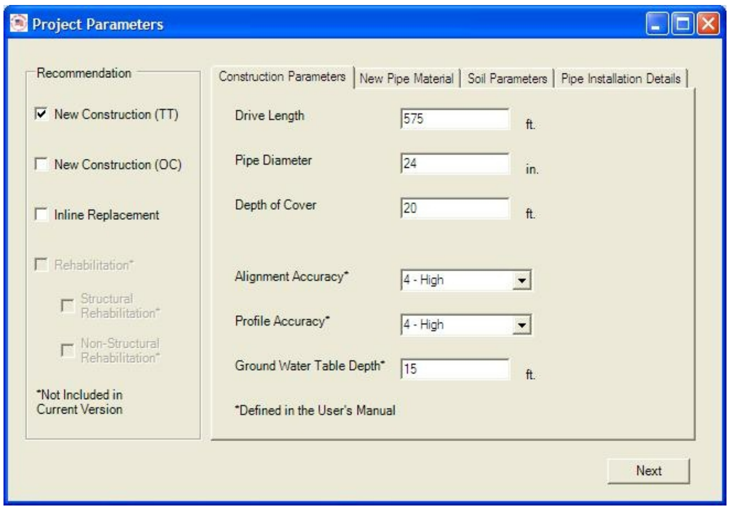


Figure 3. Construction parameters input data form.

The second category of information is the material of the new pipe and the user is asked to select from a list of ten commonly used pipe materials. The third category of user input consists of soil compatibility parameters, in which the classification and extent of dominant soil type(s) along the proposed alignment are identified. The final category of user specified information is related to the viability of inline replacement options, specifically the extent of over sizing, material type of the host pipe and the presence/absence of excessive sagging.

Risk Analysis

Following the technical evaluation stage, methods that were deemed technically suitable for the project are then reevaluated for the relative level of risk associated with four different categories of risk. The first category is the installation parameters: drive length, pipe diameter and depth of cover. In this category the project specific values are compared to the limits of each of the remaining construction methods. This comparison results in a percentage which is then assigned a risk level based on the percentage ranges shown in Table 1.

Table 1. Risk classifications for the installation parameters

Risk Classification	*Range*
(1) Very Low	(0.00 – 0.40)
(2) Low	(0.41 – 0.60)
(3) Meduim	(0.61 – 0.75)
(4) High	(0.76 – 0.90)
(5) Very High	(0.91 – 1.00)

For example, if a project has a drive length of 280 feet, a pipe diameter of 12 inches, and a depth of cover of 21 feet and one of the technically viable methods is Pipe Bursting (Static), then the risk classification will be assigned according to Table 2.

Table 2. Sample risk classifications for installation parameters

Risk Factor	*Project Data*	*Limits of Static Pipe Bursting*	*Risk Percentage and Classification*
Drive Length	280 feet	400 feet	70% - Medium
Pipe Diameter	12 inches	36 inches	33% - Very Low
Depth of Cover	21 feet	50 feet	42% - Low

The second category of risk is the assessment of the compatibility of a given construction method with anticipated geological conditions. Geological conditions were divided into ten categories, with soil types been further quantified in terms of the number of blows per foot (as per ASTM D1452-80(2000)). The geological conditions considered by TAG are: soft cohesive soils ($N < 5$), firm cohesive soils ($5 < N < 15$), stiff-hard cohesive soils ($N > 15$), loose cohesionless soils ($N < 10$), medium cohesionless soils ($10 < N < 30$), dense cohesionless soils ($N > 30$), gravel, cobble and/or boulders, sandstone and bedrock. The compatibility of each construction method with the ten soil classes is designated in the database as either: fully compatible (Y), possibly compatible (P), or incompatible (N). The rules of acceptance or rejection are given below:

1. If one or more of the dominant soil types is considered incompatible (N) with a given method, the method is deemed not permissible and is eliminated from further consideration.
2. If all geological conditions were found to be compatible (Y) with the construction method in question, then the method is considered to be permissible and the associated level of risk is considered to be very low.
3. If all geological conditions were found to be possibly compatible (P) with the construction method in question, then the method is considered to be permissible and the associated level of risk is considered to be very high.
4. If geological conditions were found to be a combination of compatible (Y) and possibly compatible (P) with the construction method in question, then the method is considered to be permissible and the associated level of risk will ranged from very low to very high depending on the percentage of length of the alignment of the possibly compatible soils, as shown in Table 3.

Table 3. Risk classification for soil compatibility

Risk Classification	*Percentage of Possibly Compatible Soils*
(1) Very Low	(0.00 – 0.10)
(2) Low	(0.11 – 0.40)
(3) Meduim	(0.41 – 0.70)
(4) High	(0.71 – 0.90)
(5) Very High	(0.91 – 1.00)

The third category of risk is the SET Index, which takes into consideration the availability of specifications, owner's experience with a given method and the method's track record. Each of these parameters has three possible "values" which are summarized in Table 4.

Table 4. Break down of the SET Index scores

Score	*Availability of Specifications*	*Owner's Experience*	*Method Track Record*
1	None	None	< 2 Years
2	Local/Regional	Some	2 to 5 Years
3	National (ASTM)	Extensive	> 5 Years

The risk classification of the SET Index is based on the sum of the score for the three parameters, which is calculated based on the user selected values, which range from a minimum of 3 to a maximum of 9. The associated risk levels are defined in Table 5.

Table 5. SET Index risk classifications.

Risk Classification	*SET Index Number*
(1) Very Low	9
(2) Low	7 or 8
(3) Meduim	5 or 6
(4) High	4
(5) Very High	3

The final category of risk contains two distinct factors, site accessibility and environmental impact. Each method has an assigned risk value in the database for environmental impact based on the potential for ground settlement and heave (potential damage to paved surfaces, nearby utilities and foundations), erosion, removal of trees and flora, creation of temporary hazards (i.e. open trenches) and migration of drilling fluids to the surface. Environmental impact is one of the six primary risk factors which are used in the calculation of the Initial Risk Analysis Index Number (RAIN). The other five factors are: (a) length ratio; (b) diameter ratio; (c) depth ratio; (d) soil compatibility; and, (e) SET index. All of which were previously discussed. Before the calculation of the Initial RAIN a weight (0.1 – 1.0) is assigned by the user to each of the six factors, by adjusting the position of a sliding scale on the screen. These weights are automatically normalized by the program.

After the weights have been assigned for each of the six primary risk factors, an Initial RAIN calculation is performed. The Initial RAIN calculation is shown in Equation [1].

[1] $$IRAIN = (LR \times w_{LR}) + (DR \times w_{DR}) + (HR \times w_{HR}) + (SETI \times w_{SET}) + (SCI \times w_{SC}) + (EI \times w_{EI})$$

Where IRAIN is the initial risk analysis index number, LR is the risk score for length ratio, DR is the risk score for diameter ratio, HR is the risk score for depth ratio, SETI is the risk score for the SET Index, SCI is the risk score for soil compatibility index, and EI is the risk score for environmental impact. The letter 'w' represents a weighting factor and the subscripts w_{LR}, w_{DR}, w_{HR}, w_{SETI}, w_{SCI} and w_{EI} stand for the length ratio, diameter ratio, depth ratio, SET Index, soil compatibility index, and environmental impact, respectively. After computing the IRAIN value, the risk score is adjusted for the degree of site accessibility, with the user selecting one of five site accessibility scenarios (e.g., limited access only), each associated with a particular gamma value (ranging from -1 to 3). The gamma value and the IRAIN are then substituted into Equation [2] to compute the final risk score:

[2] $$RAIN = IRAIN\left(\frac{1+e^{-\gamma^{Y}}}{1+e^{-\gamma}}\right)$$

RAIN is the risk assessment index number, γ is a factor reflecting the level of access to the area over the installation and the value of Y is given by the following expression:

[3] $$Y = \frac{(IRAIN-1)}{4}$$

The Risk Assessment Index Number (RAIN) is the final risk value given by the program for each technically viable method. The final step consists of a form which displays each technically viable method and its RAIN score. The user is then able to make an educated decision about which method is best for their particular project.

Model Validation

The above described model was validated by comparing its recommendations with the construction method(s) adopted by experienced design engineers for five wastewater related construction projects. The project key attributes (i.e., model's input data) are presented using a table format along with the construction methods selected by the designer and the recommendations provided by the model. All case histories presented herein were completed successfully.

Case History #1: Southside Sewer Relief Program, Edmonton, AB (gravity line)

Table 6. Data Summary: Southside Sewer Relief Program

Length	280'
Depth	21'
GWT Depth	14'
Host Pipe Diameter	8"
Host Pipe Material	Vitrified Clay
New Pipe Diameter	12"
New Pipe Material	PVC or HDPE
Alignment Accuracy	5 – Very High
Profile Accuracy	4 – High
Soil Types	Firm Clay (50%); Medium Sand (30%); Gravel (20%)
Extent of Excavation Allowed	Continuous Excavations
Site Accessibility	Medium Accessibility (Residential)
Method Selected by Designer	**Pipe Bursting**

TAG Risk Analysis Results

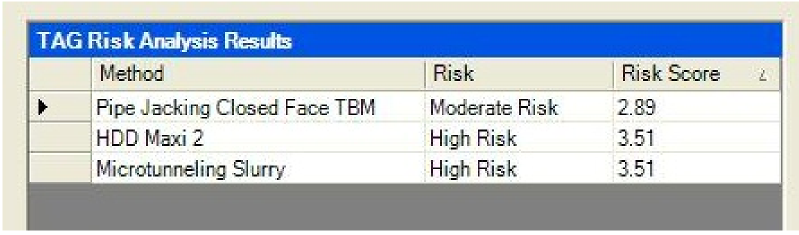
Method	Risk	Risk Score
Pipe Bursting Pneumatic	Low Risk	1.85
Pipe Bursting Hydraulic	Low Risk	1.85
Open Cut Excavation	Low Risk	2.24
Pipe Bursting Static	Low Risk	2.24
Pilot Tubing	Moderate Risk	2.78

Figure 4. Case Study #1 Results from TAG.

Case History #2: Crossing the Sacramento River, Sacramento, CA (force main)

Table 7. Data Summary: Crossing of the Sacramento River.

Length	700'
Depth	40'
GWT Depth	2'
New Pipe Diameter	42"
New Pipe Material	GFRP, HDPE, VCP and RCP
Alignment Accuracy	3 – Medium
Profile Accuracy	3 – Medium
Soil Types	Sand (60%); Stiff Clay (35%); Gravel (5%)
Extent of Excavation Allowed	Access/Receiving Pits Only
Site Accessibility	No Accessibility (River Crossing)
Method Selected by Designer	**Closed Face TBM**


TAG Risk Analysis Results

	Method	Risk	Risk Score
▶	Pipe Jacking Closed Face TBM	Moderate Risk	2.89
	HDD Maxi 2	High Risk	3.51
	Microtunneling Slurry	High Risk	3.51

Figure 5. Case Study #2 Results from TAG.

Case History #3: J. Edward Drain Interceptor Project, Westfield, IN

Table 8. Data Summary: J. Edward Drain Interceptor Project.

Length	575’
Depth	20’
GWT Depth	15’
New Pipe Diameter	24”
New Pipe Material	VCP
Alignment Accuracy	4 – High
Profile Accuracy	4 – High
Soil Types	Medium Sand (40%), Soft Clay (35%), Gravel (25%)
Extent of Excavation Allowed	Access/Receiving Pits Only
Site Accessibility	Limited Accessibility (Golf Course)
Method Selected by Designer	**Microtunneling (Slurry)**


TAG Risk Analysis Results

	Method	Risk	Risk Score
▶	Microtunneling Slurry	Moderate Risk	2.81

Figure 6. Case Study #3 Results from TAG.

Case History #4: San Francisco Zoo, San Francisco, CA (gravity)

Table 9. Data Summary: San Francisco Zoo.

Length	550’
Depth	18’
GWT Depth	12’
New Pipe Diameter	36’
New Pipe Material	RCP & PCP
Alignment Accuracy	4 – High
Profile Accuracy	4 – High
Soil Types	Loose Sand (70%), Medium Sand (30%),
Extent of Excavation Allowed	Access/Receiving Pits Only
Site Accessibility	Limited Accessibility (Zoo)
Method Selected by Designer	**Microtunneling (Slurry)**

TAG Risk Analysis Results

Method	Risk	Risk Score
Microtunneling Slurry	Moderate Risk	2.60
Pipe Jacking Closed Face TBM	Moderate Risk	2.60

Figure 7. Case Study #4 Results from TAG.

Case History #5: Bradshaw Drain Interceptor Project, Sacramento, CA

Table 10. Data Summary: Bradshaw Drain Interceptor Project.

Length	1500’
Depth	20’
GWT Depth	20’
New Pipe Diameter	108”
New Pipe Material	RCP
Alignment Accuracy	4 – High
Profile Accuracy	4 – High
Soil Type #1	Stiff Hard Clay (40%), Dense Sand (40%), Medium Sand (20%)
Extent of Excavation Allowed	Access/Receiving Pits Only
Site Accessibility	Medium Accessibility (Residential)
Method Selected by Designer	**Pipe Jacking, Open Face TBM**

TAG Risk Analysis Results

Method	Risk	Risk Score
Pipe Jacking Open Face TBM	Low Risk	1.85

Figure 8. Case Study #5 Results from TAG.

Discussion and Conclusions

TAG is a fully computerized algorithm for the evaluation of competing construction methods capable of installing, repairing, or replacing buried pipes and utilities. This approach emphasizes simplicity and practicality, while limiting the input data to that which is readily available to municipal and utility engineers during the design phase via the utilization of an extensive built-in database. A built-in wizard as well as an extensive database is used to assist users who have limited experience with trenchless construction methods.

The model was verified by comparing its predictions with the actual methods utilized in five case histories; each involving a utility construction project completed using a trenchless method in Canada or the United States over the past 10 years. These cases represent a wide range of soil conditions, pipe materials, pipe diameters, construction environments and end uses. In all cases the model identified the method used in the construction project as a viable construction method, and ranked it as either the most or second most preferred method. It is worth noting that in two of the cases (cases #3 and #5) only one method was found to be technically viable, and in one case history (#2) two methods were found to be technically viable. Thus, under many set of circumstances TAG can assist in eliminating non-viable methods and focusing the designer attention on the one or two viable methods, thus conserving costly design and planning times. Furthermore, in projects where there is a relatively large number of suitable construction methods (i.e., case #1); the model was able to distinguish between doable, but risky methods, and low risk methods. The model risk score is an absolute rather than a relative value. Thus, in case history #2 (crossing of the Sacramento River), all options were deemed risky, reflecting the technically complex nature of the project. A high risk assessment for all viable methods can serve as an indicator for the need to retain the services of a specialized design firm and/or the need for more extensive design effort.

Complications in the course of a utility construction project are nearly always attributed, at least partially, to inadequate or incomplete data during the design and bidding phases. By performing TAG's construction methods evaluation procedure the designer earns an appreciation of the type and extent of the needed data and their importance in determining the suitability of various trenchless methods. This alone is a worthy reason to justify the utilization of TAG in every municipality and design office dealing with the installation and rehabilitation of utilities.

One of the reasons for the ability of the software to closely identify the most suitable construction methods for actual projects is the close cooperation between the TTC and the Trenchless Subcommittee of the National Utility Contractors Association (NUCA), which resulted in a successful marriage of proper decision making algorithms and hundreds of years of combined experience in utility construction. Currently the authors are working on an updated version that will consider in addition to the current methods 16 new pipeline rehabilitation methods (manhole-to-manhole and spot-repair).

Acknowledgment

The authors would like to thank the National Utility Contractors Association (NUCA) for providing financial support. Special gratitude is extended to NUCA's Trenchless Subcommittee, under the chairmanship of Mr. Brandon Young, which spent numerous hours ensuring that the capabilities and limitations of the various construction methods are properly reflected.

References

ASTM D1452-80(2000). Standard Practice for Soil Investigation and Sampling by Auger Boring.

Davies, R.. Alloway B. and Fernando S. (2005). "Dispute Resulting from Horizontal Directional Drilling Through Difficult Ground," 2005 No-Dig Conf., Orlando, FL, April 24-27, NASTT, Arlington, VA.

Gokhale, S., Makarand H. and Abraham, D. 2000. "A Decision Model for the Evaluation of Options for Sewer Pipeline Installation: Microtunneling Case Study," 2000 No-Dig Conf., Anaheim, CA, April 9-12, NASTT, Arlington, VA.

STEEL WATER PIPE – THE IMPORTANCE OF FABRICATOR CERTIFICATION

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ABSTRACT

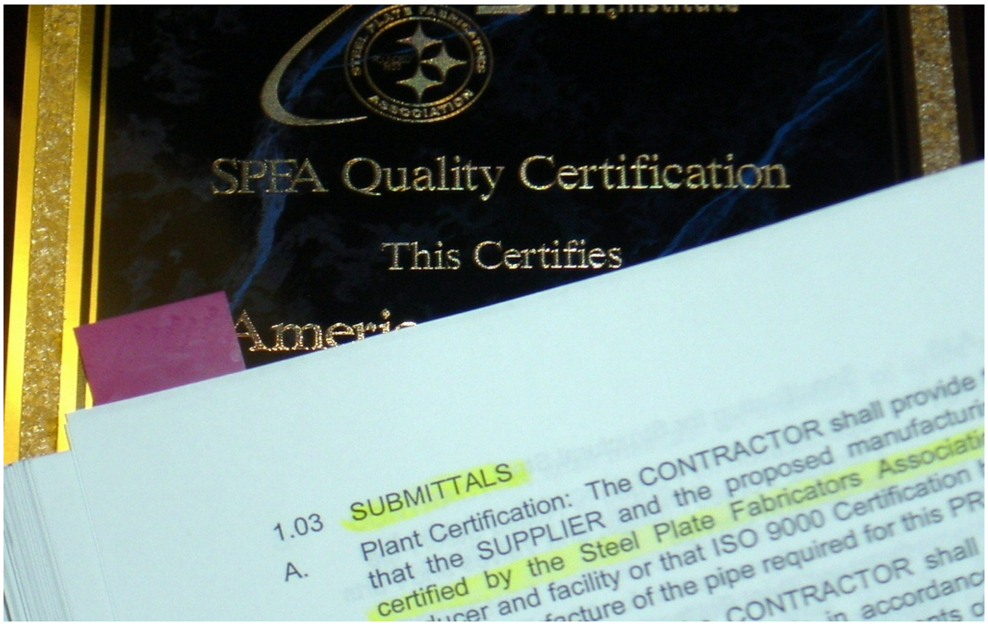
Specifying engineers already know the many benefits of fabricated steel pipe in the water transmission industry. The flexibility to design steel pipe to meet the needs of any project, in the most demanding of conditions, has evolved fabricated pipe into a highly focused industry.

Water pipe customers should insist upon a quality product and at least minimum industry standards in their contract documents. Current industry requirements are trending toward quality steel pipe certification from a third party testing organization. These certifying agencies need to be familiar with quality manufacturing requirements.

This discussion will look at the importance of fabricator certification. For example, what traits are desirable in a certifying agency? Why are third party auditor qualifications so important? What attributes/processes should be audited at the fabrication facility? And what kind of follow-up systems are needed to insure compliance?

In addition, the discussion will include applicable industry standards for steel pipe and subsequent quality requirements, as well as related accessories, linings and coatings. Process control, fabrication/welding, testing, inspection and documentation will also be included in the discussion.

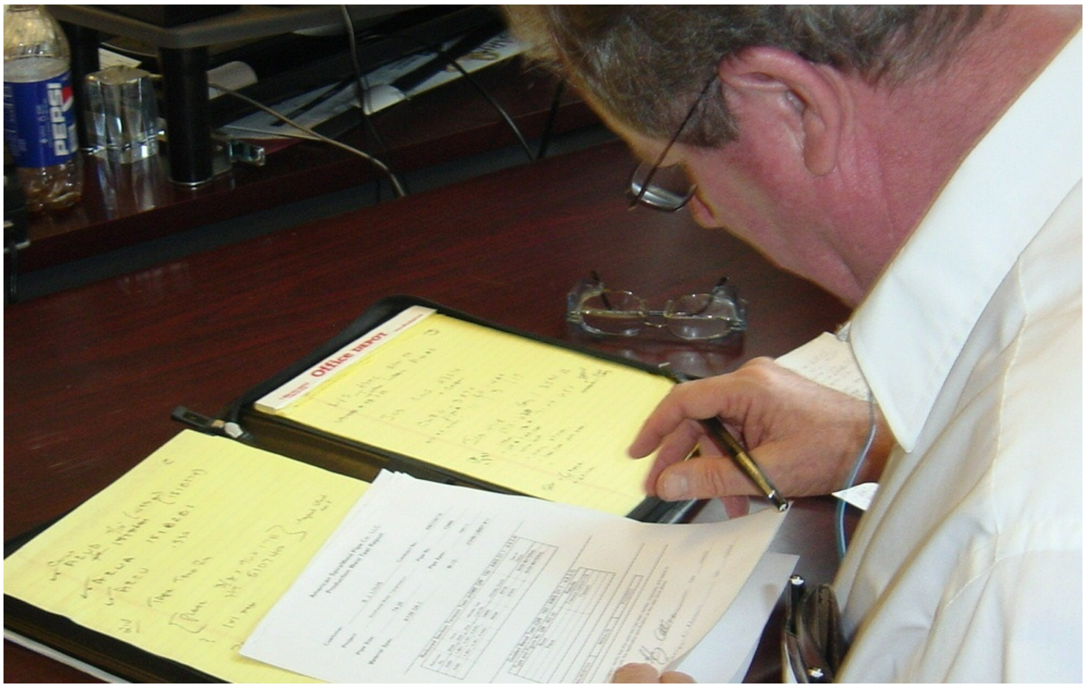
The Steel Plate Fabricators Association (SPFA) developed a certification program for steel pipe fabricators almost 20 years ago. The Steel Pipe Section of STI/SPFA is a collection of many of the steel pipe and fitting manufacturers that supply product to the water transmission industry, as well as many of the suppliers to that industry. Many specifying engineers recognize the SPFA certification program today as an essential part of their project specifications. A brief overview of the SPFA pipe manufacturer certification will complete the discussion.



Why Certification?

Why would you want a certified pipe manufacturer? Certainly there are many manufacturers out there that can fabricate and deliver a quality product without any certifications. So why do they need to be certified? It is a matter of confidence and consistency. Are you sure that your getting the correct materials, do their testing procedures follow specifications, are their pressure gauges and test instruments calibrated? If there is a problem a few years down the road, what assurances do you have that the gauges were calibrated and that the material met specification?

When you contract with a manufacture who has third party certification, you would certainly expect that their gauges are calibrated and materials meet specification. Third party certification often provides peace of mind that you are dealing with a quality organization that not only meets but also exceeds specifications and industry standards. A manufacturer may have the qualifications and certifications to supply pipe to certain standards but without third party review are you really sure?

Water pipe customers and consulting engineers often specify third party certification in their contract documents because they want a supplier with a proven quality control system. They insist that the supplier they choose for the contract has demonstrated (through a certifying agency) the ability to meet the requirements of the standard. If they did not require third party certification, evaluation between non-certified and certified supplier's quotations would result in an unbalanced playing field.

Third Party Agencies

When selecting a pipe manufacturer does it really matter who performs their third party certification? If they are certified, why should I be concerned with who certified them? The answer to the first question is yes, you most certainly want to know who the third party agency is and what qualifications they have to justify them as a reputable organization. In response to the second question there are many third party agencies that will gladly provide their services regardless of their knowledge or qualifications. A third party agency is not going to accept any liability for their certification services. I can almost guarantee that the agency's contract will have specific language to this effect.

So what should one look for in a third party certifying agency? First, I would look for an organization that is familiar with the pipe industry and an organization that has a proven track record and is highly regarded in the service sector. Often third party agencies themselves are certified in accordance with industry standards. Do their assessor's have the correct training, experience and certification to perform audits? The SPFA certification program for example, subcontracts professional qualified assessors. Do they have a good audit checklist and is it applicable to the type of industry being certified? Was the checklist created by the manufacturer, with the intent of promoting oneself or was it created by the third part agency, that is not familiar with steel fabricated pipe? These are just a few things you should ask yourself when looking at third party certifying agencies.

Auditors/Assessors

Probably the most critical element in the certification process is the lead auditor/assessor. This individual has the task of assessing the manufacturer as it relates to the audit checklist. This person must determine if the manufacturer is in compliance with the checklist. This involves reviewing operation and quality control manuals, test data, certification records and material test reports just to name a few. This also entails an understanding of the issues relating to that particular industry.

A good assessor understands the checklist and what is necessary to provide objective evidence that the manufacturer is in compliance. The assessor should be able to determine if there is a weakness and if further investigation is necessary. The assessor must be able to determine if the evidence is credible, meaning it is not just something the manufacturer produced that may or may not be reliable documentation. The assessor must determine if an audit finding has been identified, based on the checklist requirements.

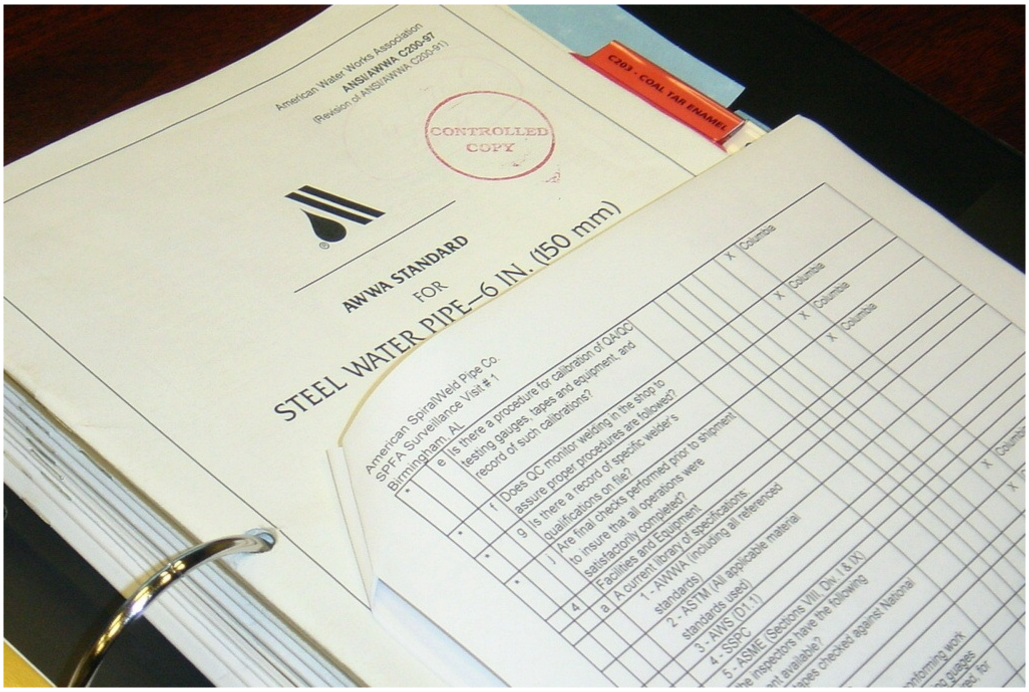
A good assessor will have the necessary training and experience to perform audits. When we talk about training, we want the assessor to have experience performing audits under the supervision of a lead assessor. There are a number of reputable firms that perform this type of training and/or certification but there is no replacement for on the job experience. Preferably, the assessor will have some previous employment history in the field of manufacturing that is being assessed. That includes welding, coating and testing knowledge, since we are talking about steel pipe fabrication. You would expect that the assessor has had experience with these types of trades.

Checklist

What attributes/processes should be audited at the fabrication facility? A good evaluation checklist will encompass the entire manufacturing process. It should be broken down to specific areas such as:

- Management
- Engineering & Drafting
- Operations & Procurement
- Coatings & Linings
- Quality Control

The checklist must include questions related to the organizational structure of the manufacturer, including job descriptions and duties. Engineering and drafting requirements to insure that the correct shop drawings and specifications are employed must be included. Operation and procurement must be reviewed to insure they meet specification requirements. Coating and lining procedures must be checked for compliance with application performance requirements. Finally, a review of quality control and assurance relating to inspection test procedures and records, including non-compliance, must be performed.



A detailed audit checklist will insure that all significant attributes/processes are evaluated. Determining what is significant is also key. Expertise in pipe manufacturing is critical in determining the components of the complete process needing to be evaluated and controlled. The SPFA checklist for example, was created, reviewed and updated by a consensus process, including steel pipe manufacturers, third party assessors and SPFA technical staff.

Follow-up

So the manufacturer has been certified by a third party agency, how long is the certification good for? 1 year, 2 years, 5 years? It is always a good idea to have a system in place to maintain accountability of the manufacturer. A lot can change within a company over a period of 2 to 3 years. People retire or move to other positions. Markets change, sometimes for the better (or worse) and standards and specifications are always changing. To simply certify a company and not follow up with them is not accomplishing the goal. Therefore the program must have conditions defined that pursue manufacturers to ensure that they are maintaining the requirements of the original certification.

Industry standards

Fabricated steel pipe has a large number of standards which manufacturers produce and certify their product. Many owners and engineers rely on the American Water Works Association (AWWA) for their standards on water pipe. These standards have critical features to insure the finished pipe is suitable for its intended use. But what are some of those critical features? Is it the hydrostatic leak testing requirements? Do welders or welding operators need certification? Or is it the type or grade of steel required for certain applications? In addition, do elbows, tees, reducers or other in line fittings that have specific requirements? Often these items are very critical during field installation.

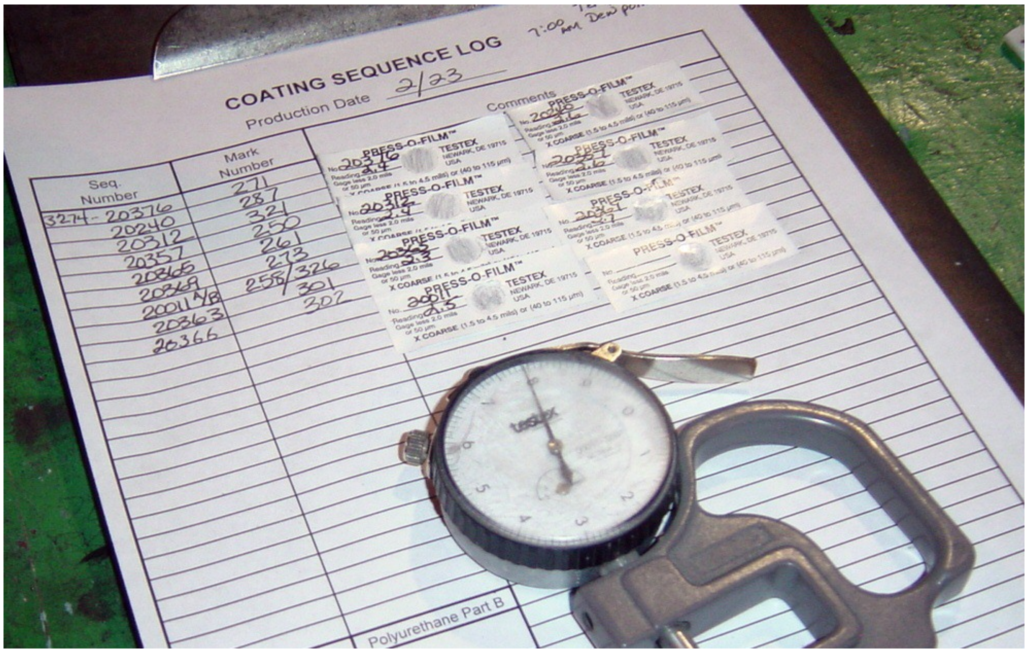
For instance, AWWA requires welders to be certified, but understanding the certification process can be very confusing to some individuals. The standards often include references to other to standards as an option for welder certification. This is why a Certified Welding Inspector (AWS QC1) is required to be employed by the pipe manufacturer in the AWWA C200 standard ("Steel Water Pipe-6 In. and Larger").

Often steel pipe applications call for a protective lining and AWWA C205 standard ("Cement-Mortar Protective Lining and Coating for Steel Water Pipe-4 In. and Larger – Shop Applied") is specified in the contract document. Critical tests identified in the standard require the manufacturer to examine the lining material for conformance. Pipe manufacturers must have properly calibrated test equipment to conduct these types of tests or have a reliable vendor (laboratory) capable of performing the test for them. Along with the testing, the pipe manufacturer must have well documented testing procedures to insure accurate results.

The AWWA C200 standard permits nondestructive testing of special sections. The use of radiography or ultrasonic test methods requires a very specialized skilled technician. The technician must be certified for the given method and have proper testing procedures to conduct the nondestructive examination.

Process Control

Process control, whether it is in welding, testing, inspection and/or documentation is always a critical feature. Does the manufacturer have qualified welding procedures? Does the manufacture perform non-destructive testing and if so, does he have the proper inspection procedures and personnel capable of detecting unacceptable discontinuities? What about hydrostatic test fixtures, are they adequate? All of these processes need control to insure compliance with the specification.



You cannot inspect quality into the pipe! You either have it ingrained in you process or you don't. No inspection or testing is going to improve the pipe. When you apply proper process control to the manufacturing process you develop quality.

Often when there is a problem, an adjustment may be made to a piece of equipment, e.g.: a welding machine. So the problem goes away, but just about the time you forget about it, the problem returns. Two choices can be made then, 1) go back and adjusting the welding machine again to temporarily fix the problem or 2) investigate what is actually causing the problem to permanently fix it. Simply put, ***control of process = better quality***!

SPFA Certification program

The Steel Plate Fabricators Association (SPFA) pipe certification program is one of the most recognized third party certification program in the water transmission industry. Applicants go through a rigorous audit with over 100 individual checklist items every three years. After the initial audit, every year in between, our third party assessor returns for a follow-up that includes over 50 individual checks.

The checklist, as mentioned previously, is a consensus document produced by the manufacturers, assessors and SPFA. Who better knows the industry (as it relates to fabrication) than the manufacturers themselves? The checklist identifies all of the most critical AWWA standard requirements. All the necessary testing, inspection instruments, welding, coating & lining, and personnel certification requirements, as prescribed by the standard are included in the SPFA checklist.

The assessors of the third party organization have many years of experience in steel fabricated pipe and the SPFA certification program. Assessors are assigned to the plant by the third party agency, so the manufacturer can not just arbitrarily choose the assessor of their liking. Our third party agency, Lloyd's Registry Quality Assurance, Inc. has an excellent reputation in the inspection/auditing service industry.

Information regarding the SPFA Pipe Quality Certification Program can be obtained by visiting: www.steeltank.com. The following companies have earned the designation and are certified under the SPFA Pipe Quality Audit Certification Program:

- American SpiralWeld Pipe Company, LLC.
  Columbia, SC
  Birmingham, AL
- Ameron International – Water Transmission Group
  Phoenix, AZ
  Tracy, CA
  Etiwanda, CA
  Fontana, CA
- Continental Pipe Manufacturing Company
  Pleasant Grove, UT
- Hanson Pipe & Products, Inc.
  Grand Prairie, TX
- Mid-America Pipe Fabricating & Supply
  Scammon, KS
- RTLC Piping Products –
  Kosse, TX
- Skyline Steel
  Cartersville, GA
- Trinity Steel & Pipe
  Weir, KS

References:

American Water Works Association (AWWA), 6666 West Quincy Avenue, Denver, Colorado 80235

American Welding Society (AWS), 550 NW LeJeune Road, Miami, Florida 33126

Temporary Diversion Systems: Reliability Is <u>Everything</u>

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ABSTRACT: Trenchless gravity sewer upgrade projects, whether through CIPP (cured in place pipe), pipe bursting, etc., tend to have one thing in common: the need for reliable pumping systems. Whether the source is overflow, collapse, construction, etc., gravity sewer projects typically require flow diversion. Once a gravity sewer line's flow has been diverted, reliable pumping is essential throughout the remainder of the project so that sewer contractors can devote their attention to the sewer upgrade.

Introduction

Temporary diversion systems are only as successful as their planning and coordination. Data collection is critical to sizing pumps appropriately and should include: line size, flow (peak and wet weather), suction manhole depth, discharge location, system run time, and noise level. This paper presents an overview of the terminology and data collection necessary before contacting a diversion pump company. Proper application of system knowledge can eliminate most major causes of failure or delay.

Understanding System Components

Properly planned diversions rely on an understanding of critical components. A temporary diversion system includes a surface-mounted, portable, automatic self-priming solids handling pump. In order to size the correct pump for the application, you need to ask the following questions about existing conditions:

- What is the line size to be diverted?
  Gravity sewers range in diameter from 8" upwards of 108" with most common sizes between 12" and 36".
- How much is the flow? (peak, low flow)
  The following questions need to be answered during the planning process if this information is not available from the wastewater treatment plant monitoring systems:

- What is the daily peak flow?
  Gravity sewer flows vary during the day with typical peak times between 6 - 9 AM and 6 - 9 PM.
- What is the minimum flow?
  Gravity sewer flows usually have a low flow period, typically mid-day or the middle of the night. Sizing a pump to too big for these periods can create short cycling of pumps (i.e., too many starts and stops).
- What is the peak wet weather flow?
  Storm water may enter lines through cracks, dramatically increasing flow during rain events.

- How deep is the suction manhole?
  Surface mounted self-priming pumps can lift product up to 28' vertically. You need to remember to account for the height to the eye of the impeller.
- How far is the discharge location?
  The horizontal distance and vertical rise (if applicable) are measured to determine friction and gravity resistance in the discharge piping.
- Are we discharging into an existing forcemain? If so, what is its size? Length? Change in elevation? Are there other stations discharging into it?
- Will the system run constantly?
  Consider fueling and refueling during a diversion. You may need to make use of an additional fuel tank to keep the pump(s) running continuously.
- Is noise an issue?
  Are you in a residential neighborhood? If so, consider making use of a pumping system that offers noise abatement.

Determining flow
The planning and sizing scope of the diversion starts with the need for accurate flow data from the municipality or engineering firm involved in the project. To understand flow data, you need to be familiar with the following terms:

- *Flow* - Million Gallons per Day (MGD); approximately 700 gallons per minute (gpm)
- *Velocity* - Feet per second; typical design is for 2' per second in gravity sewer pipes
- *Slope* - % expressed as a decimal; 1% slope would be expressed as .01
- *% Full* - 1/4, 1/2, 3/4, full

In addition to the questions listed in *Understanding System Components*, consider the following information when determining flow.

- What is the slope of the line?
  The slope of the sewer and the % full the line is flowing affects the volume of sewage flowing through a gravity sewer line (see Figure 1).

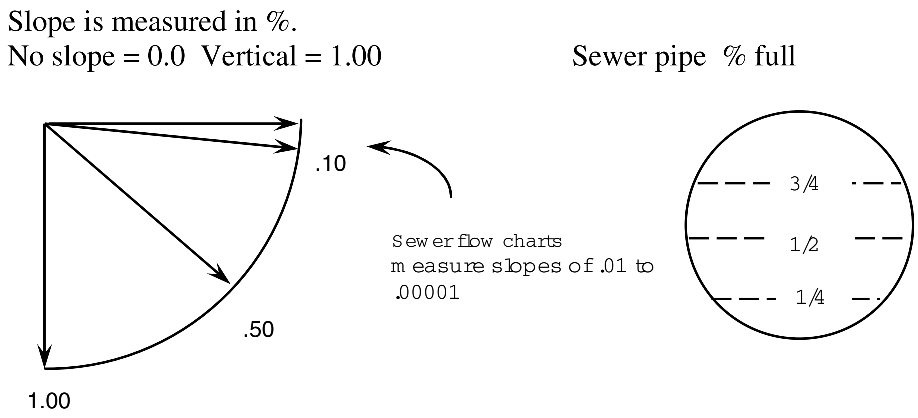


Figure 1. Determining line slope.

- What percent full is the line in average, daily and peak wet weather flow periods?
- What velocity is the sewage at average, daily and peak wet weather flow periods?
  Measure the distance (in feet) from one manhole (A) to the next downstream manhole (B). Gauge velocity <u>during peak flow periods</u> by dropping a floating object into a manhole A and timing it as it flows to downstream manhole B. The result is feet per second.
  Product pumped through a line travels at a certain velocity. The greater the velocity, the higher the flow.
- How much wet weather infiltration does the line experience?
  Stormwater may enter lines through cracks, dramatically increasing flow during rain events. Stormwater infiltration is difficult to determine and often requires some measure of estimation.
- If the suction manhole is exceptionally deep, can the suction manhole be safely surcharged and to what depth? Or, is a submersible pump required?
  To answer this question, you need to determine the elevation upstream to the first gravity line that empties into the manhole. This information is important because you do not want the flow backing up into homes, schools, offices, etc.

Taking all of these factors into consideration, you can begin to determine flow. The formula used to determine flow in a pipe or hose is:

$$[d(in.)]^2 \times vel.(ft/s) \times 2.45 = flow(gpm) \qquad [1]$$

where 2.45 is derived by converting diameter (expressed in inches) and velocity (expressed in ft/s) into flow (expressed in gpm).

In pumping applications, *area* refers to a pipe or hose (cylindrical object). The area of a circle is determined as follows:

$$A = \pi r^2 \qquad [2]$$

where π is 3.1416 and *r* = the radius of a circle (which is ½ the diameter).

Since pipe is measured in diameter, *r* can be substituted with

$$\frac{d}{2} \qquad [3]$$

Therefore, r^2 =

$$\left(\frac{d}{2}\right)^2 \text{ or } \left(\frac{d \times d}{2 \times 2}\right) \qquad [4]$$

which can be expressed as $\frac{d^2}{4}$ and substituted for r^2 in the formula

$$A = \pi r^2 \qquad [5]$$

The revised equation for area is expressed as:

$$A = \frac{\pi[d(in.)]^2}{4} \times Vel. \frac{ft.}{s} = gpm \qquad [6]$$

however, multiplying square inches by feet and dividing seconds does not produce gallons per minute. Therefore, some conversion factors involving gallons and minutes need to be added to the equation to solve for gallons per minute. The revised equation with conversion factors is:

$$\frac{\pi[d(in.)]^2}{4} \times Vel. \frac{ft.}{s} \times \frac{7.48 gal.}{1 ft.^3} \times \frac{60 s}{1 \min.} \times \frac{1 ft.^2}{144 in.^2} = x \frac{gal.}{\min.} \qquad [7]$$

For example, for a 6" pipe with liquid flowing at a velocity of 12 ft./s, you could determine flow as follows:

$$6" \times 6" (d^2) \times \ 12 ft/s \times 2.45 = 1{,}058 gpm \qquad [8]$$

By adding the conversion factors, like units of measure can be cancelled out of the numerator and denominator.

$$\frac{\pi d^2 (\cancel{in.}^2)}{4} \times Vel. \frac{\cancel{ft.}}{\cancel{s}} \times \frac{7.48 gal.}{1 \cancel{ft.}^3} \times \frac{60 \cancel{s}}{1 \min.} \times \frac{1 \cancel{ft.}^2}{144 \cancel{in.}^2} = x \frac{gal.}{\min.} \qquad [9]$$

leaving

$$d^2 \times Vel. \times \left(\frac{3.1416 \times 7.48 \times 60}{4 \times 144} \right) = gpm \quad [10]$$

Simplified, the fraction

$$\left(\frac{3.1416 \times 7.48 \times 60}{4 \times 144} \right) = gpm \quad [11]$$

yields

$$\left(\frac{1409.05}{576} \right) = 2.4462673 = 2.45 \quad [12]$$

The following flow rates are based on sewers flowing 50% full at 2 ft./sec (see Table 1).

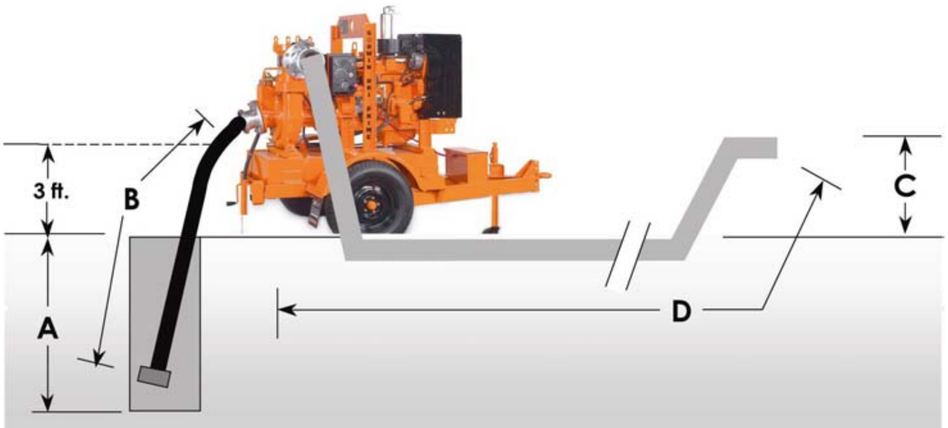
Table 1. Flow rates for sewers flowing 50% full at 2 ft./sec.

Line size	Flow (MGD)	Flow (gpm)
12"	0.5	350
18"	1.2	805
20"	1.4	980
24"	2.0	1,960
36"	4.6	3,220
48"	8.0	5,600
72"	18.5	12,950

Sizing the Pump for the Application

When determining the correct pump for an application, take the following into consideration (see Figure 2):

- Desired capacity in U.S. gallons per minute.
- Static suction lift (vertical height from the water to the pump).
- Static discharge head (vertical height of discharge hose/pipe).
- Size, type and length of any pipe or hose and fittings.
- Pressure desired at discharge point, if any.



A = Maximum depth of suction point to ground level
B = Length of suction hose required
C = Vertical elevation difference from pump to discharge point
D = Length of discharge hose/pipe required

Figure 2. Typical Horizontal Open Discharge Pump System Design.

With an understanding of these parameters, you are ready to size the pump for the application (see Figure 3).

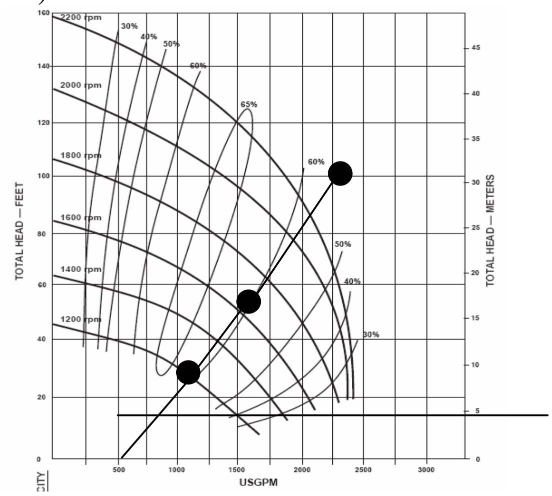


Figure 3. Determining the engine speed for the desired flow rate.

1. **Vertical elevation difference (static lift) on suction and discharge = __15__ ft.**
   Note: Static <u>suction</u> lift must not exceed 28 feet for Dri-Prime® pumps.
2. **Draw a horizontal line across the performance curve representing the static lift.**
   Diameter of Suction and Discharge hose = 8"
3. **Length of Suction and Discharge hose = 2030 feet**

4. **Fittings:** **Qty.**

90° Bend =	20 ft x	3	=	60	ft.
Check valve =	50 ft x	______	=	_______	ft.
Tee =	40 ft x	______	=	_______	ft.

5. **TOTAL EQUIVALENT LENGTH OF HOSE = 2090 feet**

6. **Multiply friction loss by equivalent hose/100, then add static for TDH.**

Flow (gpm)	Friction		Hose			Static lift						
500	0.5	x	2090	ft/100	=	11	+	15	ft	=	26	TDH
1,000	2.0	x	2090	ft/100	=	42	+	15	ft	=	57	TDH
1,500	4.1	x	2090	ft/100	=	86	+	15	ft	=	101	TDH

7. **Plot duty points at 500, 1000 and 1500 gpm. Determine engine speed to accomplish desired flow rate for the application.**

Consider All of the Options

No doubt that calculating data helps make an informed decision when selecting the right pump for your application. However, there are some options you have that only experience can help you take into consideration. When choosing the correct pump, take these options into consideration:

1. Practical experience shows that it is very difficult to achieve discharge velocities in excess of 12'/sec. For example, based on the 12'/sec. rule of thumb, the following flow rates can be achieved in various suction and discharge pipe sizes (see Table 2):

   Table 2. Flow rates bases on pipe diameter.

Diameter	Approx. flow rate (gpm)
4"	500
6"	1,000
8"	2,000
12"	4,000

2. Pump selection – Try to pick a minimum of two pumps to handle the peak flow. Never try to use one large pump to match the peak flow event. Think of a lead-lag-backup set up that you might find in most permanent lift stations. This avoids putting the entire bypass on one machine, and will avoid short cycling of a large pump. This is before adding a standby or back up pump to the selection. Use the minimum or low flow to drive the pump size selection. All pumps on the bypass should be the same size to ensure redundancy.
3. Variable speed pump controls – Whether electric- or diesel-driven pumps are selected, the technology is available to automatically vary pump speed to match flow and or variable head conditions. This is especially useful on multiple pump configurations on long discharge pipelines. The pumps can vary their speed as changing flow conditions increase or decrease the TDH.

This keeps fuel or electrical costs to a minimum and increases the reliability of the pumps through the duration of the bypass pumping period.

4. Noise abatement – Will you be pumping in a residential neighborhood? If so, you should consider sound attenuation on your bypass pump. Sound attenuation can reduce noise to 69 dB at 30 feet. Consider combining sound attenuation with the use of float controls on the main pumpset, so that the backup pump only runs [quietly], when necessary. Variable speed controls will also reduce the pump speed during low flow periods, also reducing noise levels.
5. Freeze protection – Are you planning your diversion for the winter months? Is freezing a common occurrence? If so, make provisions for freezing conditions by accounting for continuous flow and continuous drainage in your diversion system. Those adjustments should prohibit freezing. In addition, consider equipping your backup pump with a trickle charger so that your battery does not drain. Finally, block heaters go a long way in ensuring the dependability of diesel engines in the winter months.
6. Traffic flow – Are you working in a residential area? Would you prefer to avoid tearing up roadways? If so, consider using a road ramp to divert flow in low-traffic, low speed areas.
7. Auxiliary fuel tanks – Will you require your pumps to run continuously? If so, how long can your pump run on a full tank of gas? Consider an auxiliary fuel tank for continuously running operations.
8. Plan for an online backup pump – To ensure continuous pumping during a diversion, a backup or standby pump is used as a redundant spare. A control panel activated by floats in the suction manhole automatically starts the backup pump as necessary. Though admittedly redundant, the backup pump is especially applicable in longer-term projects.
9. Telemetry or alarms – Is there an existing SCADA system that the temporary bypass pump controls can be tied into? Or will a temporary automatic alarm system be implemented? Consider a wireless autodialer to warn of high flow events or unexpected shutdown of pumps during off hours.

Summary

Temporary diversion systems are only as successful as their planning and coordination. Data collection is critical to sizing pumps appropriately. You should be aware of the terminology and application of data before contacting a diversion pump company. Your role is to understand that:

- Sewer bypass applications require continuous, reliable pumping during repairs or upgrades. In order to achieve reliable pumping, remember:
    - When considering the operating speed of pumping equipment, faster is not always better. Use the recommended speed from the pump supplier and prepare a refueling schedule based on the estimated fuel consumption.
    - Use of automatic float controls on pumps serves as a safety precaution in emergencies by initiating backup pumping when necessary.

   - If you encounter pumping problems, the most common pump parts to check are the non-return valve, suction screen plug and volute drain valve (which may be open).
   - You need to decontaminate the system prior to dismantling. Be sure to make provisions to flush out the pump and piping, and drain and flush connections by hooking into a fire hose or other fresh water source.
- Pump systems must be sized and designed to handle peak flows during dry weather conditions.
- Surface-mounted solids-handling pumps are a more efficient means of providing reliable bypass pumping.
- Detailed specifications for the bypass application are critical to the successful completion of a bypass.

The appropriate pumping specialist will help you by:
- Conducting a job-walk to determine logistics, flows, related issues (discharge piping run, noise)
- Using a complete bidding specification (if required) for diversion pumping.
- Reviewing your diversion pumping plans carefully.

Sewer flow can be unpredictable; thereby rendering backup plans tricky, if not impossible. Proper planning and sizing help to eliminate most major causes of failure or delay.

Carbon Fiber Liner Quality Control For Repair of PCCP

Heath Carr[1]

ABSTRACT

The safe and reliable Installation of carbon fiber liners has many critical steps. When a single step is missed, the lining installation will suffer significantly. Development of a usable quality control standard for material selection, design, installation, and inspection are critical for a successful installation. Work in this area began in the 1980's with studies conducted by the University of California San Diego for the Department of California Transportation (Cal Trans) for the purpose of retrofitting highway bridge columns. The technology moved to the general building industry following the guidelines of ACI 440.

During the mid 1990's this technology entered the pipeline industry for the retrofit of Prestressed Concrete Cylinder Pipe (PCCP) with broken wires. Initially, a quality control standard directly related to this application was unavailable. Initial methods drew from the column wrapping methods developed by Cal Trans, ICC and those outlined in ICC (International Code Council) AC 125. However, as this application became more popular both the Fyfe and Fibrwrap Construction companies made large advancements in materials, application, and quality control methods.

In the year 2000, the Metropolitan Water District of Southern California, MWD, took a serious interest in carbon fiber liners for the retrofit of its 163 miles of PCCP and implemented a testing and review program for CFRP (carbon fiber reinforced polymer).
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Presented in the paper are the most recent methods developed for the safe and reliable installation of carbon fiber liners. Discussion of mobilization concerns, materal requirements and related verification testing, critical paths for installation, and inspection methods for both during and post installation are provided.

A case history is discussed outlining program considerations for both owners and contractors. Discussion is also provided outlining the pitfalls owners should avoid in order to obtain an installation within their predetermined shutdown window. Areas of discussion are: access, dewatering planning, permitting, design coordination, mobilization, contract requirements, and field support.

QUALITY CONTROL DOCUMENTS

As the development of FRP's for use in repair and strengthening of structures advanced in the early 1990's, there became an industry requirement for quality control and quality

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assurance standards and techniques. This requirement resulted in many studies and collaborations between manufacturers, universities, building code approval authorities and public agencies to create inspection and testing criteria. At the forefront of this was the California Department of Transportation, who performed the initial proofing and qualifications of various FRP systems.

Caltrans developed and set forth criteria for manufacturers to qualify their FRP systems for use on public Department of Transportation projects, specifically to retrofit bridge columns for seismic deficiencies. This criteria included large scale structural testing of columns, durability testing of the materials, requirements for manufacturer quality control procedures and requirements for inspection of the installation procedure.

The cooperative research and criteria set forth by Caltrans led the way for other approval agencies to establish quality control and quality assurance documents. Criteria was establish by ICC AC 125 and AC 178 inspection criteria, and ACI (the American Concrete Institute) authored the ACI 440 guideline. There were various other criteria and documents established for the QA and QC of FRP systems, however these procedures were slow to be adopted and standardized which led to project specific requirements, typically developed by the Engineer of Record in conjunction with manufacturer specifications.

In the mid to late 1990's research was performed to strengthen PCCP by applying FRP internally to resist both internal pressure and external loading conditions due to deficiencies caused by the failure of the pre-stressing wire.

Commercial applications soon followed, and once again, the FRP industry was in need of a Quality Control/ Quality Assurance standard for the strengthening of PCCP and other pipeline structures with FRP's. Some of the early QA/QC and other FRP approval documents, such as ICC AC125, ACI 440 and Caltrans, were utilized to pre select and approve manufacturers' systems and set forth project specific quality control documents. This approach fell short of an industry standard and many agencies were skeptical of using FRP's for lack of QA/QC standards, performance history and proof testing.

Metropolitan Water District of Southern California (MWD) implemented a research program to address these issues in 2000. The program consisted of large scale structural testing, durability testing, manufacturer approval and qualification, quality control/quality assurance standards, inspection criteria and training and FRP design considerations. This program was completed in late 2002 and MWD began to use FRP to strengthen a portion of their PCCP system. This Pre-Qualification, QA/QC program has proven to be very thorough and has since been adopted by other agencies and FRP installation companies.

INSTALLATION PROCEDURE

The field installation of FRP (fiber reinforced polymer) Composite Systems must be performed correctly in order to make certain that the goals the engineer has designed for are met. In the pipe, the surfaces are prepared for bonding by means of abrasive blasting using grit or a water-blasting system to achieve a 1/16" minimum amplitude. All contact surfaces are then cleaned and dried in preparation for the composite to be applied. Using a roller, one prime coat of the manufacturer's epoxy is applied and allowed to cure for a minimum of one hour. Any uneven surfaces left from the blasting are filled in with the manufacturer's thickened epoxy using a trowel.

The epoxy matrix is prepared by combining components at a ratio specified by the system manufacturer. The components of epoxy are mixed with a mechanical mixer until uniformly mixed. The dry fabric is saturated with the saturate epoxy using a saturator machine. The machine monitors the epoxy to fiber ratio. The saturated fiber is layed up by hand inside the pipe. The FRP applied to the wall of the pipe is troweled smooth to remove any air or excess epoxy behind the fabric. Thickened epoxy is troweled on in between layer to ensure strong interlaminate bond. The number of layers applied, butt splice width, end joint details, and overlaps are all done according to the project shop drawings. All seams and edges are coved with thickened epoxy. After proper cure times are met, the final epoxy coat is applied by roller or trowel and then cured.

INSPECTION METHODS

The Metropolitan Water District of Southern California together with Fibrwrap Construction, Inc. developed a standard for field inspection and quality control on FRP strengthening for Pipes. The methods of inspection appear in all of MWD's standard specification for FRP. Each step in the installation process has a check to ensure that it was performed correctly.

In order to keep track of materials used, installed, and submitted for testing, the Contractor is required to keep and maintain a quality control log. In this log the contractor will record the lot numbers of the fabric and epoxies used, calculated fabric to epoxy weight ratios, test samples made, and locations of the material (i.e. lot numbers corresponding to layer number installed in the pipe). This data should be recorded in the log book every shift and should include dates and times respectively. This log is the contractor's means to re-create the quality control measures taken in case there is any dispute. The owner's inspector should keep a similar log so that any inconsistencies can be addressed.

The environment inside the pipe where the layup is to be performed is the first onsite test to be performed. The surface of the substrate the FRP is to be applied to must fall in between 35 and 100 degrees Fahrenheit. If the surface temperature falls outside of this range, then corrective measures should be taken to control the atmosphere inside the pipe.

Surface preparation is important to verify to ensure that the FRP material will bond properly to the substrate in order to transfer the loads properly through the bond line as designed for in the engineering stages of the project. The ASTM D4541 *Standard Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers* cannot be performed in a pipe because there is no flat surface to perform the test on.

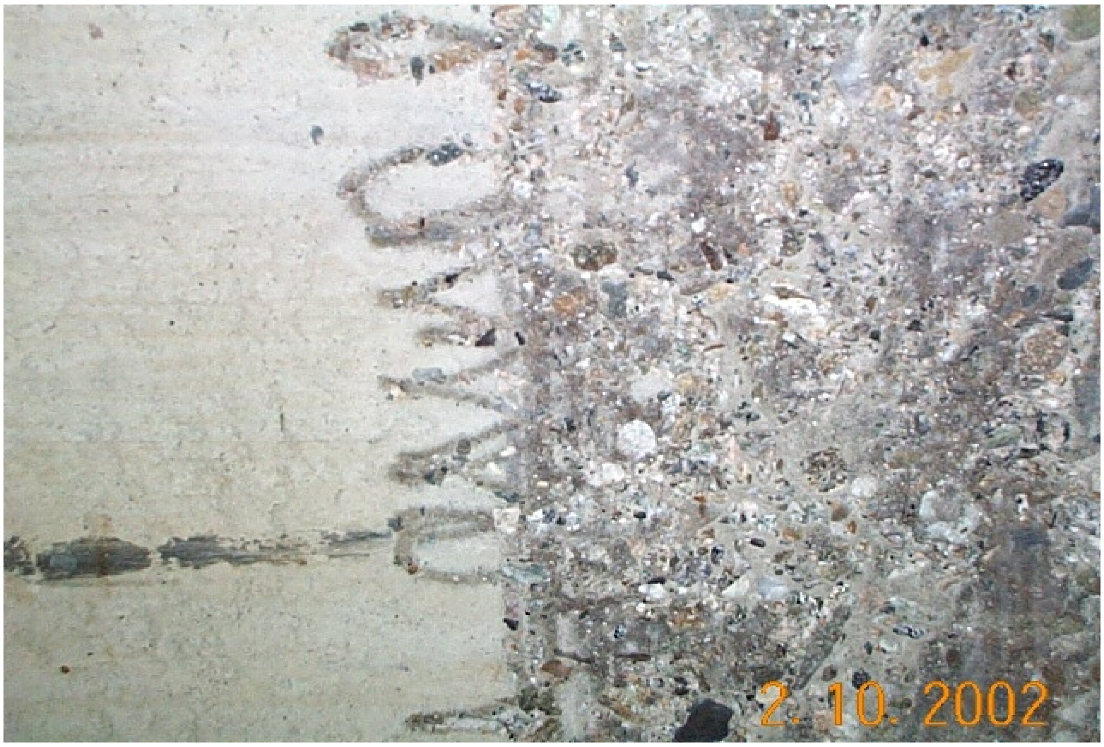

Figure 1 - surface preparation to pipe wall

Therefore all inspection of the surface preparation must be visual. The inspector is looking for a roughened surface with minimum 1/16" amplitude. A visual clue that the surface preparation is completed accordingly is the absence of any smooth surface or other latent still remaining. In most cases the surface preparation will expose aggregate of the mortar lining. Figure 1 shows a concrete surface that has an acceptable surface preparation next to a concrete surface that has not yet been prepared. The surface must also be free from fins or sharp edges that could create voids behind the fiber.

Following the surface preparation, the substrate must be dry before the prime coat of epoxy is applied. If the prime coat of epoxy is specially formulated for a wet surface prime, then the substrate may still be damp when applied. If the epoxy has not been formulated this way, then proper measures should be taken to ensure that the substrate is dry before application. The most simple way to, yet still effective, to test for moisture, is to press four layers of tissue paper against the substrate for with the thumb and hold for approximately 10 seconds. If the tissue paper is dry and shows no signs of wetness, then the pipe surface is ready to be primed. The tissue test should be performed at different locations, a minimum of 10 locations for a spool of 20 foot length. If moisture is found on the tissue, then the pipe will need further heat and dehumidification. Space heaters can also be used to dry isolated locations that still have moisture.

Material:

All epoxies prepared for priming, saturation, and final coats must be monitored for correct mix ratios. The epoxies used onsite for the FRP strengthening are two part epoxies consisting of a part A and part B. The epoxies should come from the manufacturer in pre-weighed containers that are have the correct ratio for the part B to be poured directly into the part A and mixed, without further measurement. If the epoxies are not sent pre-metered by the manufacturer, rather in bulk 55 gallon drums, then the buckets should be weighed on a calibrated scale onsite to achieve the proper ratios as set by the material manufacturer's quality control manual, prior to the mixing of the two parts. The weighing of the units should be recorded and noted in both the contractor's field log and the inspector's report. It is also important that all materials that are brought to the site are checked that they have not exceeded their shelf life.

The fiber that arrives onsite at the project must first be inspected to confirm that the fiber sent to the project is in acceptable condition. The "curl" test is used to verify proper material weave. The roll of fiber is rolled out and the edges are examined. The edges should not curl up on the fiber if the material is to be determined acceptable. The fiber should also be examined that the tows are well stitched together, and not pulling apart.

Fiber saturation is controlled by the saturated fabric test. The saturator's rollers are set at the correct spacing per the manufacturer's recommendations for the material being used. The saturated fabric test is performed to ensure that the gap has been set properly and that the proper epoxy to fabric ratio is obtained. A test piece of fabric, two feet in length, unsaturated, is cut and weighed to the nearest one hundredth of a pound on a calibrated scale onsite, as seen in figure 2.

Figure 2 - weighing the saturated material

The test piece is then run through the saturator, seen in the background of figure 2. The now saturated test piece is placed on the scale and weighed. The epoxy to fabric ratio is then calculated and should be within +/- 5 percent of the manufacturer's written recommendation. This test should be performed at the start of each wrapping shift. If the epoxy to fiber ratio does not fall within the allotted variance, then the saturator is to be re-gapped and the weigh test run again until the proper ratio is obtained.

Periodic visual inspection should take place during the course of the Fiber Liner installation. The inspector should look for the orientation of the fiber with respects to the pipe wall. The pieces oriented in the circumferential should not vary more than ½" over 12" or 5 degrees from the aligned axis. The inspector should also note that the end details are correctly done, and that thickened epoxy is being applied between the layers. The visual inspection also incorporates checking for imperfections in the materials such as rips, voids or air bubbles behind the material. The gaps between consecutive bands should not exceed ½" width in the fabric's transverse joint. The inspector should also note that the proper overlaps, as detailed in the project drawings, are being achieved. Throughout the layup process the inspector should check the materials being sent to the strengthening location and the application process.

Testing

Following the installation and cure, the fiber liner should be checked for defects, bubbles, de-laminations, or fabric tears. The inspector has a couple of methods for examining the surface. First, a hard tool such as a quarter or a marble can be hit against the installed composite. The inspector should scan over the surface of the pipe at various locations listening for hollow spots in the liner. The hollow sound implies a void behind the material. A more advanced method of this test is to use a mobile ultrasonic inspection system, like one pictured below in figure 3. The ultrasonic inspection unit is scanned over the surface of the fiber liner, and the users looks for variances in the readings to pinpoint defects.

Figure 3 - inspection for "bubble trouble"

If voids or de-laminations are found from the test then repairs are recommended. Defects less than three inches in diameter should be injected with resin or backfilled For large defects, greater than three inches in diameter. The engineer should recommend an alternate repair depending on what type of defect has occurred and where in the composite liner it has occurred.

Arguably the most important quality control measure is the laboratory testing performed following the completion of the installation. During the course of the project, samples 12" by 12" or 12" by 24" and 1 or 2 layers thick, dependent on the type of material used, are made for testing purposes. Two samples are made for each wrapping shift, one sample is given to the inspector, and one sample is retained by the contractor. A predetermined percentage of the samples made are submitted to an approved testing laboratory. The ASTM D 3039 *Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials* is performed on the samples that were made onsite as seen in Figure 4. This test verifies the mechanical properties of the material applied in the field meet those properties used in the design of the pipe strengthening.

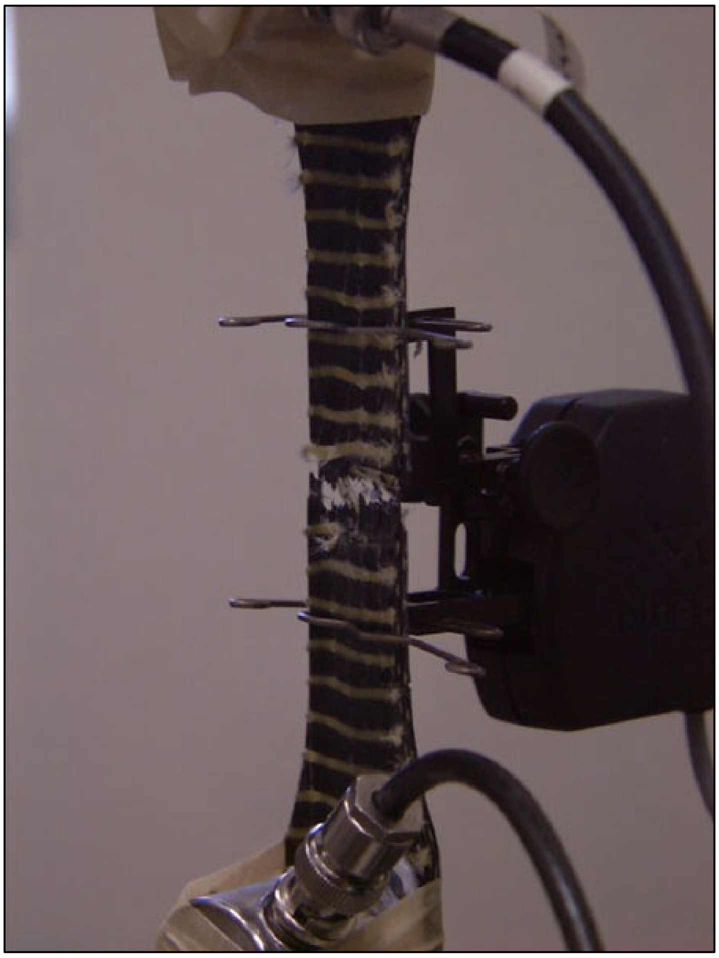

Figure 4 - ASTM D 3039 Tensile Test

PLANNING (Material Selection)

There are many quality control measures that can be taken before the project ever begins. Material qualification, Contractor qualification, stringent specification, detailed drawings, and shutdown plan, are all keys to preparing for a successful project and eliminating problems ahead of time.

Manufacturer, material and installer pre-qualification are essential to a quality FRP installation. Starting with a specification that addresses the years of experience for both the manufacturer and installation contractor. A minimum of five years is typical. The specification should also detail the number of specific projects with references, for PCCP repair it is typical to require a minimum of ten documented successful installations for the installation company and a minimum of five key personnel with the same experience. These references should be reviewed as part of the Manufacturer/Installer prequalification process.

Materials Pre-qualification should consist of a durability report for the proposed system performed by an independent accredited testing facility. The report should reflect a minimum of 10,000 hours of accelerated durability under comparable conditions in which the material may be applied. In addition the materials should have been in service for a period of at least five years. Large scale proof testing of the proposed material, such as internal water pressure testing and external de-load testing should be a requirement to pre-qualification. All of the materials shall have NSF 61 certification and approved for potable water applications.

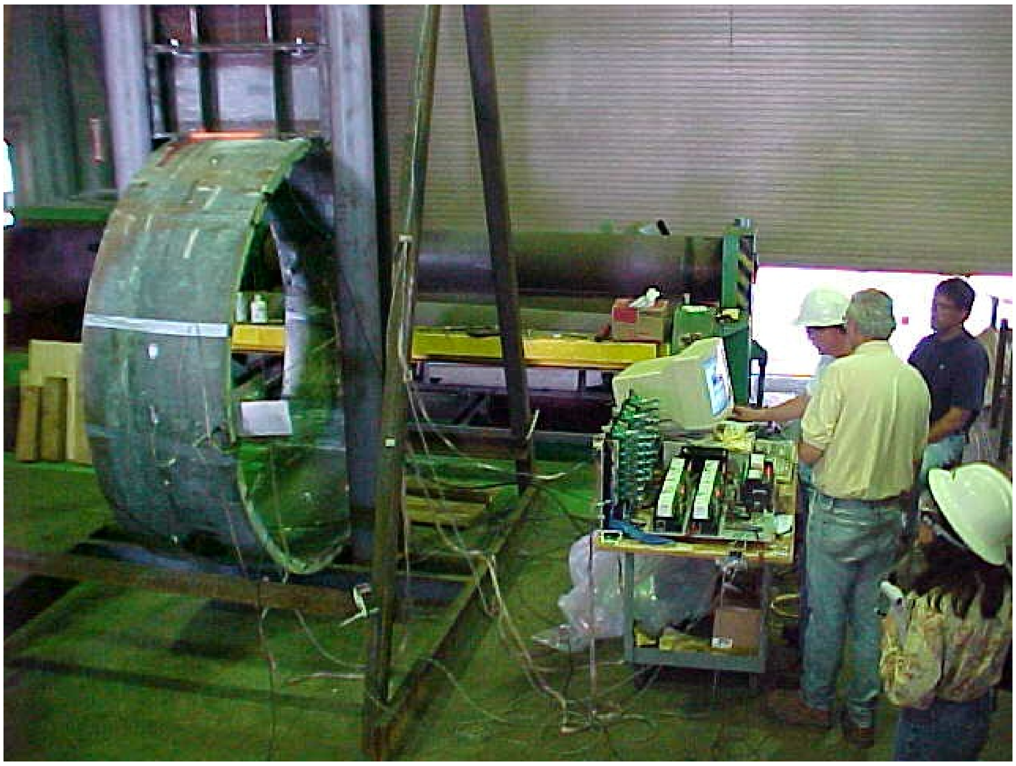

Figure 5 - external load testing at the MWD facility

Case Study of MWD Foothill Feeder

The coordination between the Municipality and the Contractor that they have hired is essential to completing the job successfully. The owner must feel confident in the ability of the contractor that they have hired to understand the correct measures that must be taken to ensure a safe project and correct installation. Likewise the contractor relies on the owner to take the necessary precautions to ensure the contractor's safety and facilitation of the work.

The Metropolitan Water District of Southern California's Foothill Feeder required spot repairs at two different locations in the line. Carbon fiber lining was chosen as the best repair method for the spot repairs. MWD then negotiated the project with Fibrwrap

Construction, Inc. a certified installer of Fyfe Company materials. Both Fibrwrap Construction, Inc. and Fyfe Company are pre-approved contractor and material supplier, respectively for MWD.

The Foothill Feeder Pipeline that required repair was a 201" diameter pipe, the largest that MWD had strengthened using FRP composites. There were many factors that posed difficulties for the project. The repair locations were 1,200 feet and 2,400 feet from access points into the line. Because of the size of the pipe and distance to access new safety standards applied that had not been encountered on previously performed carbon fiber liner repairs.

MWD feeds water to many neighboring districts. The shutdown of the Foothill Feeder Pipeline caused protest amongst many of the other agencies and utilities. MWD was forced to take these opinions to heart and limit the length of the shutdown period in order to satisfy the needs of the other distributors. Any contractor hired to perform the carbon fiber installation must have sufficient experience to perform these repairs without failure. The pre-qualification, QA/Qc program allowed MWD the confidence to set a very aggressive shutdown time length for such a logistically complicated job.

REFERENCES

1. ACI 440.2R-02: Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures, 2002.
2. CALTRANS AENC. DOC 8.11.97, 1997.
3. ICC (AC 178) Interim Criteria for Inspection and Verification of Concrete and Reinforced and Unreinforced Masonry Strengthening Using Fiber-Reinforced Masonry Polymer (FRP) Composite Systems, June 2003.
4. The Metropolitan Water District of Southern California, Specification Section 09980 Fiber Reinforced Polymer Composite Material Lining, September 2004.

Robotic RFEC/TC Inspection of Transmission Mains with Reducers: Practical Aspects

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Abstract

In 2005, a non-destructive testing evaluation was conducted on a 36-inch Prestressed Concreter Pipe (PCP) line for a large water utility in Illinois using the patented Remote Field Eddy Current / Transformer (RFEC/TC) Technology. RFEC/TC determines the number and location of wire breaks in individual PCP pipe. Due to the small diameter of this line, both man-operated bicycle-type tool and the remote-operated tethered system were used for the inspection. Data collected from the two systems were both of good quality and comparable to each other.

The utility recognizes the value of the RFEC/TC inspection and plans to conduct more inspections in other PCP sections. When possible, the remote-operated tethered system, known as PipeCrawler, will be utilized because of safety considerations and because PipeCrawler is capable of inspecting pipes in a water-filled condition.

In November 2006, the same water utility used the RFEC/TC technology to inspect approximately 4 miles of another 36-inch line. This inspection was challenging because it contained seven 36-inch by 24-inch reducers. Consequently, it was not possible to use the man-operated bicycle tool. In addition, it would not have been possible to use the PipeCrawler tool in its normal configuration, because of its fixed length mast.

To overcome this challenge, a controllable and extendable mast was designed to fit on the PipeCrawler to facilitate the RFEC/TC inspection of PCP mains with reducers. In this paper, we will discuss the practical aspects of this.

Remote Field Eddy Current / Transformer Coupling

Remote Field Eddy Current / Transformer Coupling is a condition based asset management technique that detects and quantifies the number of broken prestressing wires along the length and around the circumference of prestressed concrete pipe (PCP). This patented system is currently applicable for the condition assessment of PCP in diameters greater than 24-inch, including embedded cylinder pipe, lined cylinder pipe and non cylinder pipe.

The RFEC/TC technique involves collecting data with a tool that travels through the pipeline, and then analyzing the collected data to identify broken wires within individual pipes. Data for pipelines greater than 60-inch in diameter is collected using PipeWalker, a manned upright tool that is walked through the pipeline; data for pipelines between 36-inch and 60-inch is collected using PipeRider, a manned cycle tool; and data for pipelines (24-inch and larger) is collected using PipeCrawler, a remote controlled tool. These tools are presented in Figure 1. The recently developed PipeRanger tool can inspect lines as small as 16-inch in diameter, which extends the range of RFEC/TC inspection to all sizes of PCP manufactured.

PipeWalker PipeRider PipeCrawler

Figure 1. RFEC/TC Inspection Tools

PipeCrawler

PipeCrawler is a tethered, remote controlled, all wheel drive vehicle that can traverse up to 2000-feet depending on pipeline configuration. It was developed to facilitate the inspection of smaller diameter pipelines of which a manned inspection would be difficult. Unlike inspections using the PipeWalker or PipeRider tools where the entire portion to be inspected needs to be dewatered, for PipeCrawler, only the entry point for lines with small access points requires dewatering so that the tool can be assembled inside the pipe.

PipeCrawler is attached to an umbilical cord which supplies power to the tool and is also used to pull the tool backwards. A video camera is mounted to the tool to facilitate driving, particularly when navigating around bends. The footage also shows the visual condition of the internal pipe wall. The video footage as well as the data collected can be viewed in real time via two monitors. The umbilical cord and the monitors are stored inside a van (see Figure 2) which is used to transport the equipment to each pipeline entry point.

Figure 2. Communication system (left) and umbilical spool (right) stored in van

Project Background

In 2006, PPIC was presented the opportunity to perform an inspection on an approximately 4-mile section of a 36-inch pipeline that could not be dewatered. This section of pipeline contained seven reducers that changed the pipeline diameter from 36-inch to 24-inch. The RFEC/TC technology requires the tool to be configured relative to pipe diameter for optimal data collection. Specifically, the component located on the mast of the tool should be set at a fixed distance from the pipe wall. Since the mast height is set before the inspection and remains fixed throughout the inspection, the existence of these reducers posed a challenge.

Several options were considered to overcome this challenge. The installation of additional access points was considered. In this case, the tool would be driven up to the reducer, and then driven back to the access point where it would be removed and inserted into another access point on the other side of the reducer. This would allow the tool to then be driven to the other side of the reducer. With this technique, all reducers must be separated by an access point, which was not the case of this pipeline. To accomplish this, four additional manholes would need to be installed.

Since installing a manhole costs between $100,000 and $150,000, per manhole, the overall cost of this preparatory work was a significant factor in the considerations.

Another option was to configure the tool so that it could pass through the reducers. To do this, the mast would need to be set approximately one foot lower than the optimal setting. This would result in decreased sensitivity to wire breaks, which may cause small amounts of distress to go undetected.

Although it was preferred to avoid dewatering, performing a manned inspection using the PipeRider was also considered. This option had additional concerns as well as the undesirable dewatering factor. There is a general safety concern for manned entry of pipelines of this size with reducers. To be between two reducers without an access point makes exiting the pipeline a slow process due to the time involved in passing the reducer. To have a reducer in front of the tool, regardless of what is behind it is also a safety concern as the PipeRider is difficult for a person to pass due to the amount of space it takes up within the pipe. Passing the reducer is also a difficulty in general as the tool must be disassembled and reassembled to fit thru the space. This activity not only extends the time needed for inspection, and therefore, the time the pipeline remains out of service.

A forth and final option was reviewed and decided upon. This option involved modifying the tool so that the mast could be extended and retracted remotely. This option allowed for the inspection to be performed without dewatering; for the PipeCrawler to pass the reducers without compromising data quality; and for the inspection to be completed without the additional cost of manhole installations.

Extendable Mast

In order to modify the tool so that the mast could be extended and retracted remotely, a pivoting mast structure that mounts onto the rear of the PipeCrawler was developed and fabricated. Several design parameters were considered including the geometry that was required when the mast was in the vertical and folded positions. This was determined by conducting computer simulations of the PipeCrawler maneuvering through the reducers. Another design parameter that was met was the vibration reduction of the exciter due to the dynamic nature of the pivoting mast. This was achieved by designing a rigid mount to guide the mast and stabilizing the top with an actuator arm. The custom mounted actuator arm also provides the mechanical power in order to pivot the mast. Waterproof limit switches used to control the mast were also designed and fabricated.

Project Summary

In November, 2006, PPIC completed the inspection of the 4 mile section of 36-inch pipe. The inspection spanned a total of six days in which the data was collected for 1378 individual pipes. A total of seven reducers were encountered and successfully passed with the use of the extendable mast. When the reducers were observed via the video feedback, the RFEC/TC tool was driven up to the edge of the reducer, the mast was lowered (see figure 3), the reducer was traversed, and the mast was then raised to it's original position (see figure 4).

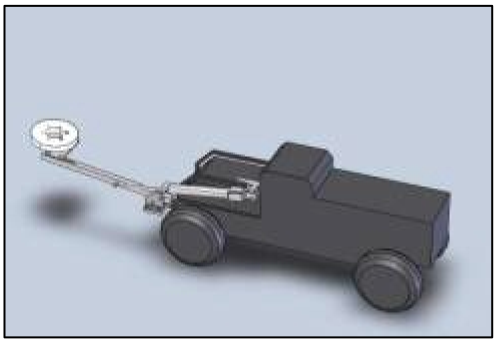

Figure 3. Mast in lowered position

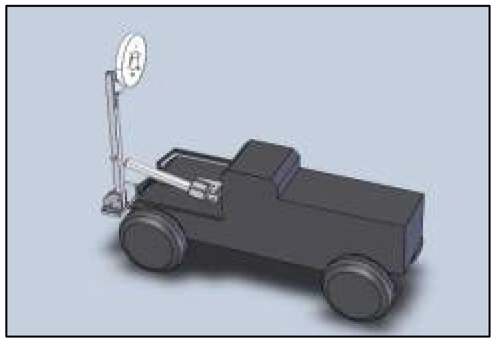

Figure 4. Mast in raised position

The data collected was of good quality, on par with the data quality of the other inspection tools. The data revealed distress in numerous pipes, which were quantified using data from calibration tests performed on pipes from this pipeline.

Conclusion

The PipeCrawler system is used to locate and quantify wire breaks in PCP 24-inch and larger in diameter. Initially, the existence of reducers, such as the ones in the pipeline described in this paper, would have resulted in either the need for additional access points as the tool cannot pass through the reducer with the mast in a fixed position, or decreased sensitivity if the mast were to be set low enough for the tool to pass the reducers.

This challenge was overcome with the development of the extendable mast that enabled the inspection to be completed without compromising data quality, and without the additional cost of manhole installations. The inspection of the pipeline in the case study presented in this paper has demonstrated that the system has been successfully adapted to maneuver through reducer laden lines. In addition to enabling the PipeCrawler to pass through reducers, the extendable mast will also aid it passing through certain valves depending on the exact configurations, and the inspection of lines with multiple diameters during a single survey, therefore simplifying the unmanned inspection process.

Sewer Pipeline Operational Condition Prediction using Multiple Regression

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Abstract

One of the key factors for better performance of sewer pipeline networks is proper monitoring of existing operational or hydraulic condition of pipes. The hydraulic performance of sewer networks involves many uncertainties and is dependent upon vulnerability and retention capacity of each pipe segment in the concerned network. Random inspections of pipes are expensive. This paper suggests an objective methodology for evaluating operational condition of pipes. A multiple regression model is developed on the basis of historic condition assessment data for predicting existing operational condition rating of sewers. The regression model produces a most likely existing operational condition rating of pipes by utilizing simple inventory data. The developed model is intended to assist municipal engineers in identifying critical segments influencing overall hydraulic performance of the system.

Introduction

The evaluation of operational performance of sewers is a serious challenge for municipal managers all across North America. In any urban drainage system suffering from surcharging and flooding at unacceptable frequencies, the key to the development of an effective solution lies in a detailed understanding of the present performance (Adams et al. 2000).

Therefore, proper condition assessment of sewers should be the first step towards achieving efficient and effective management. However, this requires an enormous amount of budget allocation and resources. Consequently, due to lack of knowledge regarding the current operational condition of sewers, sewers become susceptible to catastrophic failure, which results in flooding and loss of properties.

Research Objectives

This paper focuses principally on developing a new methodology for predicting sewer's operational performance through the use of historical data, which attain better and cost-effective management. This paper demonstrates the use of a multiple regression modeling application in order to provide decision makers with a means of prioritizing sewer sections for scheduled and detailed inspection.

Previous Research

Hasegawa et al. 1999 developed a method for condition prediction of sewers on the knowledge of pipe material, length, diameter and other characteristics. However, the procedure was complex and could lead to anomalous results (Fenner, 2000). Ariaratnam et al 2001 developed logistic models condition evaluation of sewers. The model was developed through historical data based upon factors; such as, pipe age, diameter, material, waste type and depth. Similarly Kulnadaivel (2004) developed a structural condition prediction neural network model by utilizing historical pipe inventory data.

All of the research mentioned above is limited to predicting structural condition of sewer pipelines. The operational performance of sewers is of primary importance as far as the immediate end user complaints are concerned. Sewer overflows may cause communities to be vulnerable to various health problems and other monetary losses. This puts a lot of burden on municipal managers to minimize end user complaints. Therefore, operational condition prediction would be helpful to facilitate decision makers to control sewer overflow problems and prioritize inspection and rehabilitation needs.

Factors Influencing Operational Condition of Sewers

There are several factors which could deteriorate the overall operational condition of sewers causing overflows. These factors can be divided into two categories as shown in Figure 1 (May et al. 1998):

- Non – Hydraulic
- Hydraulic

Non-hydraulic problems are generally defined as those deficiencies in sewer performance which are not due to lack of flow capacity within the sewer system. As shown in Figure 1, these problems sometimes are very uncertain, such as, for example, random blockage of flow due to some object or pumping station failure, etc. However, some factors like structure condition of a pipe are more likely predictable.

Structural condition of a sewer affects directly its flow capacity, as older pipes have rougher inner surface, more structural cracks, and breaks, resulting in more debris and reduction in diameter due to deformation. The pipe's roughness also depends upon its material. A pipe's structural condition further depends upon many factors; such as pipe age, pipe material, pipe depth, bedding material condition, soil condition, existing dead and live loads above pipe etc. Therefore, all these factors directly or indirectly have an influence on the operational condition of sewers (Adapted from Peggs, 1985).

Furthermore, non-hydraulic problems are greatly dependent upon the operational and maintenance strategies and history. Routine maintenance and repair programs could increase the service lives of sewers; however, more budget allocation would be required in this regard. Nevertheless, regular maintenance of sewers could minimize the risk of catastrophic damage (May et al 1998).

Hydraulic problems occur if the sewer is not adequate enough to sustain high volume of flow. The causes of these problems could be faulty design for pipe size and its gradient. Pipe size includes its diameter and length. Larger diameter pipes can accommodate larger volume of flow. Similarly, longer lengths of pipe mean less bends to accumulate debris creating blockage (Kulandaivel, 2004).

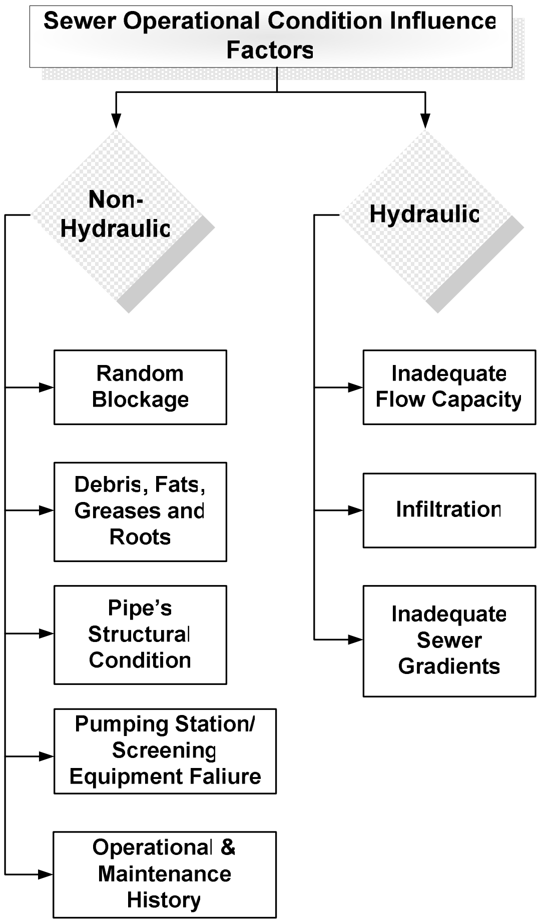

Figure 1: Major factors affecting sewer pipeline operational condition (Adapted from May et. al 1998)

Another major hydraulic factor is infiltration and inflow. Infiltration occurs when groundwater enters a sewer system through broken pipes, defective pipe joints, or illegal connections of foundation drains, while inflow is surface runoff that enters a sewer system through manhole covers and exposed broken pipe (May et al 1998).

Operational Condition Variable Attributes

As illustrated above, there are many factors which contribute to the existing operational condition of a pipe. It is impossible to consider all factors in one research program (Zayed et al 2005). Therefore, the current study concentrates only on some of the major factors that are included in the operational condition prediction model development. The variables, which are considered in developing the operational condition model, are based on the facts mentioned in Figure 1. The major non-hydraulic influence factor would be the structural condition of a pipe. However, if the structural condition of a pipe is known through some inspection records, then obviously, the pipe's operational condition would also be known through the same inspection.

Therefore, it should be a primary assumption for the operational condition prediction model development that structural condition of sewer is not known. Nevertheless, structural condition of a pipe is further dependent upon many known factors; such as, age, size, depth, soil and traffic conditions, etc. Although age related structural condition deterioration of sewers is unclear (Fenner, 2000), for simplicity it can be assumed that pipe age is a major factor contributing in a sewer's structural condition.

Regression Model Development Methodology

The methodology can be divided into two steps; data acquisition and data processing. The data acquisition part mainly consists of defining key parameters for data collection, collecting and preparing data. The main attributes affecting the operational condition have been described in previous sections.

Data regarding these attributes was collected from Pierrefonds municipality, Quebec. After collection data are prepared according to the given objectives for developing the regression model. The collected data are based on CERIU (Centre for Expertise and Research on Infrastructures in Urban Areas, Canada) classification system. CERIU protocols directly assign a condition class to each operational defect; such as, pipe deformation, debris, etc. However, it does not calculate any condition class for the whole pipe segment. Therefore, a methodology is adapted for converting CERIU classification data into WRc (Water Research Centre, UK) protocols, which clearly specify a condition class on the basis of individual defects to a sewer pipe. The condition classes vary from 1 to 5 depending upon the severity of defects; for example, class 1 for excellent condition and class 5 for the worst.

The next step was to define categorical variable "pipe material" by some quantitative measures which can be used as a predictor for the regression model. In this context, Manning's roughness coefficient was considered which is widely applied in partly filled conduits (Casey, 1992).

The Manning's roughness coefficient can be found out from the Manning's formula which is expressed as:

$$V = \frac{R^{0.67}\sqrt{S}}{n} \qquad \text{Equation 1}$$

Where

V = Mean Velocity of Flow (m/s)
n = Manning's Coefficient of Roughness
S = Channel Slope (m/m)
R = Hydraulic Radius (m) (It is the ratio of cross-sectional area to wetted perimeter)

Although there are wide disagreements between researchers on the value of n and extensive research on its determination is going on (Bilgil, 2003), the general ranges of n for various pipe materials have been defined (Gribbin, 2001), which are shown in table 1. Table 1 shows how the attribute "pipe material" was expressed in the model in terms of Manning's roughness co-efficient "n".

Table1: Manning's Roughness Co-efficient Values (Adapted from Gribbin, 2001)

Pipe Material	"n" Value Range	Input Value of "n" for Model
Concrete	0.011 to 0.015	0.011
Asbestos Cement	0.011 to 0.015	0.011
PVC	0.009 to 0.011	0.009

Regression Model Design

Regression is a data oriented technique because it deals directly with the collected data without considering the process behind it (Zayed et al 2005). As a result, the original data was carefully pre-processed according to the adapted procedures described in the previous section. For the data processing or model building part, the Minitab® statistical software package was selected. Minitab is an available powerful, flexible, and easy to use (Kulandaivel, 2004) statistical software package. The pre-processed data was divided into two parts; about 20% of data was picked randomly to be used for model validation, while the rest of the data was used for model development.

Due to broad spectrum influence of predictor variables upon response variable "Operational Condition Grade", many regression models were designed by defining different functional forms of variables. The functional forms for variables can be explained as follows:

The general form of multiple linear regression equation is defined as:

$$E\{Y\} = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \ldots\ldots\ldots\ldots\ldots.. + \beta_{p-1} X_{p-1} \qquad \text{Equation 2}$$

Linear regression model include not only first – order models in p-1 predictor variables but also more complex models. Therefore, model with transformed variables or with different interaction terms will be considered as linear regression models due to their respective linear parameters. For example, the following two equations present linear regression models (Kutner et al 2005):

$$Y_i = \beta_0 + \beta_1 X_{i1} + \beta_2 X_{i1}^2 + \beta_3 X_{i2} + \beta_4 X_{i2}^2 + \beta_5 X_{i1} X_{i2} + \varepsilon_i \qquad \text{Equation 3}$$

$$\log_{10} Y_i = \beta_0 + \beta_1 \sqrt{X_{i1}} + \beta_2 \exp(X_{i2}) + \varepsilon_i \qquad \text{Equation 4}$$

In this context, different functional forms of variables were defined for different possible scenarios, and different regression models were built through the selected statistical software package. The necessary statistical diagnostics were applied to each model for further processing and decision making and will be discussed in the next section.

The best possible relationship obtained through regression model design procedure is shown in equation 5. The units of all the parameters are according to Table 5.

$$(Operational\_Condition\_Grade)^{0.63} = \frac{0.308 + 0.567\left(\frac{Age}{Diameter^{n}}\right)(Length)^{Slope}}{(Age)^{0.63}}$$

Equation 5

Figure 2 shows the scatter plot for the regression model. The figure illustrates that the response function "Y" has been transformed through a power "λ". This transformation was done according to the Box-Cox procedure to remedy lack of fit and inconsistent error terms. The Box-Cox procedure automatically identifies a transformation from the family of power transformations on Y for which the standard deviation of Y is minimum.

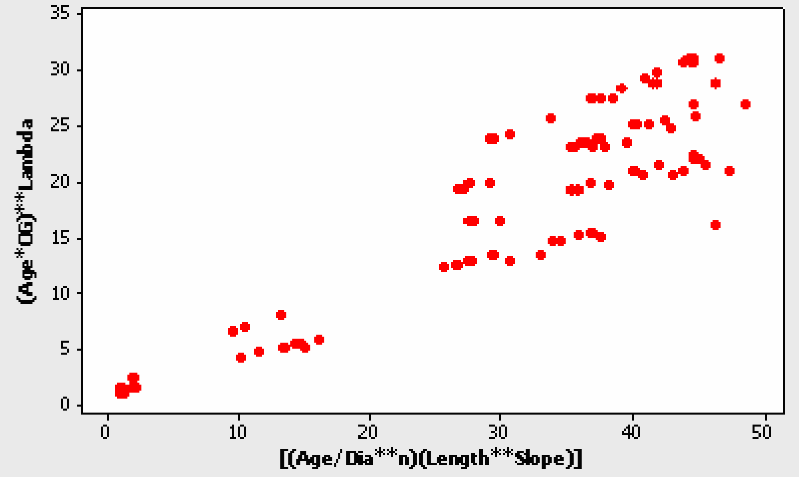

Figure 2: Scatter Plot for Sewer Pipeline Operational Condition Assessment Regression Model

Figure 3 shows the Box-Cox plot for the response function. It shows that the power transformation value λ when the standard deviation in Y is minimum is 0.63. Therefore, the response function is transformed by the power transformation of 0.63.

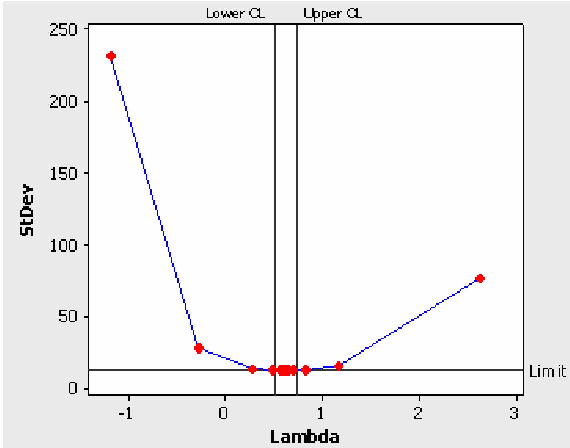

Figure 3: Box-Cox Plot for Response Function

Diagnostics

In the previous section, the developed models are statistically checked for all relevant parameters and the best form is chosen as the best fit of data in hand. The first essential check deals with identifying co-relations and interactions between parameters.

The Operational condition assessment regression model compares the relation of operational condition of sewers against its basic parameters. The interaction and correlation between theses parameters are checked to determine whether it is statistically valid to be embedded in the model. The main diagnostics in this regard are: coefficient of multiple determination (R^2), F test for regression relation (P(F)), "t" test for the validity of each regression coefficient (P(t)), and lack of fit (LOF) test.

The co-efficient of multiple determination measures the proportional variation in operational condition explained by sewer's attributes; age, diameter, material, length and gradient. The results shown in table 2 illustrate that 87.9% of the total variability in operational condition can be explained through the developed regression equation. The R^2 (adjusted) accounts for the number of predictors in the model. Both values indicate that the model fits the data well.

Table 2: Sewer's Operational Condition Assessment Model Statistical Analysis Results

R^2 (%)	R^2_{adj} (%)	P (F)	P (t)		P (F) Lack of Fit	
			Intercept	X	Pure Error	Data Sub-setting
87.9	87.8	0.000	0.007	0.000	--	0.073

To determine P(F) for the whole model, a hypothesis test is carried out. The null hypothesis (H_0) assumes that all regression coefficients, β_0 and β_1 in this case, are zero i.e. $\beta_0 = \beta_1 = 0$. The alternate hypothesis (H_a) assumes that not all of them equal to zero. Based on the Minitab's output the p-value for the test is 0.000 (as shown in table 2). This means that null hypothesis is rejected.

The next step is to determine the validity of regression coefficient individually. "t-tests" were performed separately for the β_0 and β_1. In case of β_0, the null hypothesis (H_0) of t-test assumes that $\beta_0 = 0$; while alternative hypothesis (H_a) assumes that $\beta_0 \neq 0$. Similarly, the other null hypothesis assumes that $\beta_1 = 0$ and vise versa. The results of these tests, shown in table 2, indicate that the p-value for intercept is 0.007 and for predictor is 0.000. As a result, alternative hypothesis is accepted. Note that for performing F and t tests, the confidence interval α is assumed to be 0.05; that means that null hypothesis could be accepted if the p-value is equal to or greater than 0.05.

A lack of fit test (LOF) determines whether a model best fit data or not. Minitab performs this test in two ways;

- If enough replicates are available → pure error test
- If not enough replicates are available → data subsetting test

Lack of fit test was performed on the model under consideration and it was found that the program could not perform pure error test due to lack of replicates. Consequently, data subsetting test results were displaced and p-value for F-Test was found out to be 0.073 (as shown in table 2). The decision criterion in the case of data subsetting test is that the p-value should be equal or greater to 0.01 for an ideal fit model. Therefore, the result shows that 73% of the data can be fitted through the model.

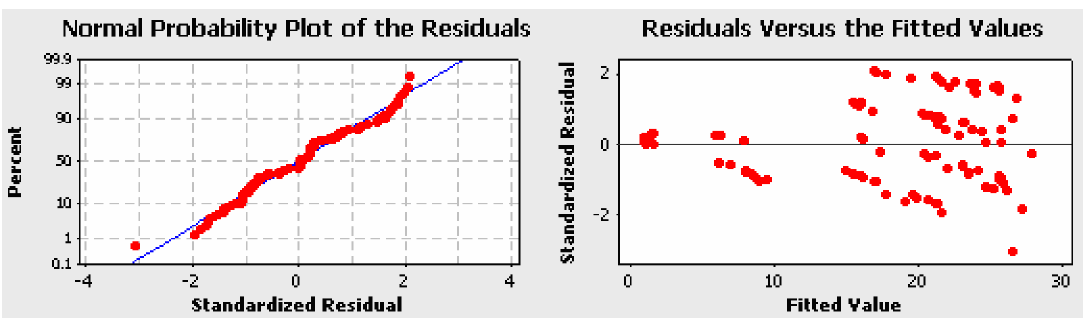

Figure 4: Residual Plot for the Regression Model

The next step is to perform residual analysis for the model. Figure 4 shows the normal probability and fitted value plots for the residuals of the developed model. The Normal probability plot shows that the distribution of residuals is almost normal. As small departures from normality do not create any serious problems (Kutner et al 2005), the results could be considered as satisfactory. However, there could be a possibility of outliers. After a careful examination of the unusual observations, it is found that the pipeline operational condition deterioration phenomenon is sometimes extremely uncertain. For example, one of the unusual observations shows that the pipe's operational condition is in class 1 (Excellent) while the pipe's age is almost 30 years. Therefore, these observations are not considered as outliers due to natural possibility of occurrences.

The fitted value plot shows a slight diagonal band pattern trend which could due to:

- Important variable(s) might be omitted from the model (Kutner et al 2005)
- Data variability issues: data composed of integer variables (Anderson et al 2005)

Both scenarios exist in this case, because important factors, such as, infiltration, etc, are not considered in the model due to unavailability of data. Further, as already mentioned, some of the variables are integers. Consequently, all the statistical analysis results, including, residual analysis, were found to be satisfactory.

Model Limitations

The regression model was developed through the obtained historical condition assessment data. The data collected for model development had some ranges for different input attributes. As a model's results will only be as good as data collected (Ariaratnam et al 2001), the model has the same limitation regarding the data variability and input ranges of predictors. Table 5 shows a brief summary of input ranges of the predictors of the developed model. As the model was validated through the data having the ranges shown in table 5; it is expected that the model will generate satisfactory results for this input range.

Effect of Age on Pipe Deficiency

As mentioned, preliminary analysis of the collected data shows that the age of a sewer has a significant effect upon its operational condition. Based on the collected data, a relation between operational condition and pipe age is developed as shown in Figure 5.

This figure shows an exponential relationship between the variables as follows:

$$Operational\_Condition\_Grade = 1.4392e^{0.022Age} \qquad \text{Equation 11}$$

Equation 11 further shows that the relationship is exponential; therefore, the operational condition deterioration rate of an older age pipe would be higher than a new one.

Table 5: Model Input Variable Ranges

Variable	Minimum	Maximum
Age (Years)	1	47
Length (m)	6	120
n	0.009	0.011
Diameter (mm)	250	1050
Bed Slope (m/m)	0.001	0.065

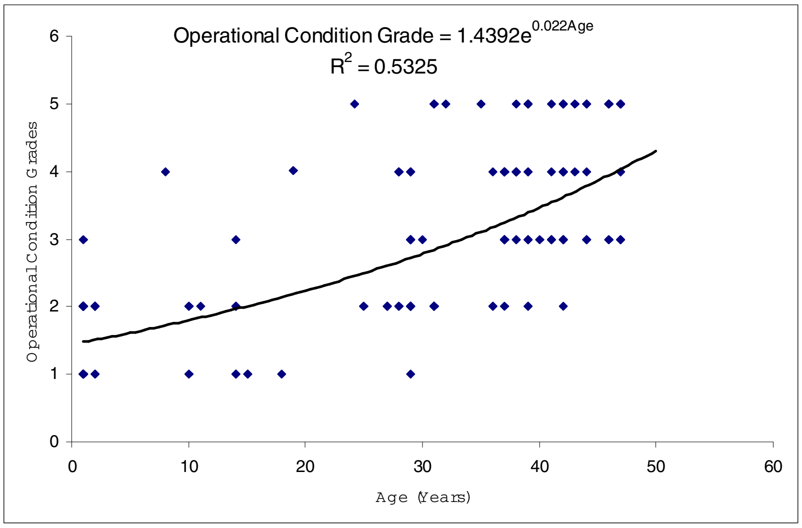

Figure 5: Affect of Sewer Pipeline's Age upon its Operational Condition

Conclusions

Although current approaches to predict sewer's condition primarily emphasize the structural aspect, the presented model in this paper provides information on the overall operational condition of sewers. This knowledge is valuable in identifying critical sewer sections vulnerable to catastrophic overflows and flooding. Consequently, this model will assist engineers in prioritizing inspection and rehabilitation of sewers and minimizing end user complaints.

During the model building process, various combinations of variables were investigated and the best scenario among them chosen for further validation. The model is validated through several approaches for minimizing the possible chance of any error. As anticipated, the historical data revealed that age has a significant effect on the operational condition deterioration of local sewer system owned by the city of Pierrefonds, Quebec.

Future research should be performed to integrate the regression model with infiltration and exfiltration phenomena. The model should also be expanded by introducing other attributes affecting operational condition, such as, operational and maintenance history. The value of Manning's coefficient of roughness was assumed to remain constant for the model development; however, this value usually higher for older pipes. Therefore, future study should include these considerations.

References

Adams, B & Papa, F, (2002), "*Urban Stormwater Management Planning with Analytical Probabilistic Models*", Wiley Inc USA

Anderson, M & Davenport, N, (2005), "*A Rural Transit Asset Management System*", University Transportation Centre of Alabama Report 04401, USA

Ariaratnam, S, El-Assaly, A & Yang, Y, (2001), "*Assessment of Infrastructure Inspection Needs using Logistic Models*", ASCE Journal of Infrastructure Systems, Volume 7, No 4, December 2001

Barqawi, H, (2006), "*Condition Rating Models for Underground Infrastructure: Sustainable Water Mains*", M.A.Sc Thesis, Concordia University, Montreal

Bilgil A, (2003), "*Effect of Wall Shear Stress Distribution on Manning Coefficient of Smooth Open Rectangular Channel Flows*", Nidge University Research Report, Turkey

Casey T. J, (1992), "*Water and Wastewater Engineering Hydraulics*", Oxford Science Publications, Oxford, UK

Fenner, R, (2000), "*Approaches to Sewer Maintenance: A Review*", Urban Water, Volume 2, Page 343 – 356, UK

Gribbin, J. E, (2001), "*Introduction to Hydraulics and Hydrology with Applications for Storm Water Managemen*t", Delmar Thomson Learning, USA

Kulnadaivel, G, (2004), "*Sewer Pipeline Condition Prediction using Neural Network Models*", Master's Thesis, Michigan State University, USA

Kutner, M, Nachtsheim, C, Neter, J & Li, W, (2005), "*Applied Linear Statistical Models*", Fifth Edition, McGraw-Hill Inc, USA

May, R, Martin, P & Price, N, (1998), "*Low-Cost Options for Prevention of Flooding from Sewers*", CIRIA Inc London, UK

Pegs, L, (1985), "*Underground Piping Handbook*", Original Edition, Robert E Krieger Publishing Company, Inc, USA

Zayed, T & Halpin, D, (2005), "*Productivity and Cost Regression Models for Pile Construction*", ASCE Journal of Construction engineering and Management, Volume 131, No 7, July 2005

Investigation of the Failures of Deep Sewers and Service Laterals in California

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Abstract

The "City" installed a sewer system as part of overall infrastructure improvements for the CFD1 project. The sewer system included 3650 m (12,000 ft) of Vitrified Clay Pipe (VCP) 20 cm (8 in) dia. to 30 cm (12 in) diameter main lines at depths from 3 to 7.3 m (10 to 24 ft), and 89 VCP 10 cm (4 in) service laterals. Within months of installation 35% of the service laterals had leaks, breaks, and misalignments. The contract identified the site conditions, extent and details of the work required to be performed in conformance with the contract drawings and specifications, the required schedule of 147 days, and the contractor's responsibilities for ensuring total quality control for the project on materials, and workmanship. The city engaged a company other than the engineer of record to provide site supervision, inspection, and monitoring similar to those outlined in the specifications. In addition, the company that had earlier carried out the geotechnical investigation was retained by the site supervision firm to provide geotechnical inspection services during construction of the project as required by the specifications.

The senior author[1] was engaged by the City's Attorneys to review the design, specifications, and construction practices including inspection and monitoring, and a current condition assessment on this project as part of an engineering evaluation of the deep sewer system. The review also involved identifying deficiencies in design, installation, inspection and monitoring, and parties responsible for such deficiencies and any negligence. This paper documents the lessons learnt.

Collection of Project Data

The City Engineer provided the following documents for review by the authors:

- Project Specifications
- Design Drawings for the Project
- Book of Photographs of Pipe Breaks - Photo Log

- Map showing the Stations where Pipe Breaks Occurred
- Geotechnical Report for the Project
- 12 Videos of Sewer Lines with Backup Documentation
- Inspection Logs for the Project

At the senior author's request, the City provided the following additional data:

- Project correspondence related to sewer and laterals installation and failures
- Backfill operation sequence
- Changes in project specifications
- Construction photographs
- Geotechnical Site Inspector's field notes

Subsequently the City provided 20 photographs of the failures at the following locations:

- Failure around a Sanitary Sewer Manhole (SSMH) at Gum Ave./Leisure Road
- Failure around SSMH at Gum Ave./Walker Street
- Failure at SSMH at the intersection of Leisure Road and Klenhard Ball Field.

Review of Geotechnical Data on the Site

A local firm carried out the geotechnical investigation program at the project site and prepared the Geotechnical Engineering Report for the project. The top soil at the site to a depth of 30 cm (12 in) is loose from agricultural operations. The upper 0.6 to 1.5 m (2 to 5 feet) of soils consist of silty clays to silty fine sands and very fine to fine sandy clayey silts. The next layer extending to depths of 3 to 4.5 m (10 to 15 ft) consisted of interbedded silty very fine to fine sands and very fine sandy silts in some areas and slightly gravelly sands and slightly silty fine sandy gravels in other areas. These layers were underlain by stiff silty clays and clayey silts. Borehole D3 differed from the above and revealed clean sand and gravels. These type of soils were later encountered during deep sewer construction and repairs of pipe and laterals as documented by site inspectors during construction as well as during repairs of the deep VCP sewers, laterals, and risers. This borehole also caved in when the auger was withdrawn.

The geotechnical investigation indicated very inconsistent, poor, difficult soil conditions particularly for the construction of deep sewers. Even though no ground water was observed even in the deeper boreholes during the site investigations, the soil was observed to be very moist near the bottom of the holes. This indicated the water table was not too deep. The highest seasonal water table which occurs in winter was reported by the geotechnical consultant to be about 4.5 m (15 ft) below the ground surface. The moist soils observed in the drillings further indicated the possibility of perched ground water.

The authors' assessment of the site is in general agreement with the findings and recommendations by the project geotechnical engineer. The site supervision firm retained by the City provided conclusions and specific recommendations for implementation during construction. These recommendations were included in the construction specifications in Section C-23 by the City. The possibility of perched ground water conditions, the seasonal fluctuation of the ground water table at the site, the occurrence of loose sands and gravels likely to be unstable during excavations were discussed in detail in the Geotechnical Engineering Report as well as summarized in the specifications. The type of soils found at the site have a high likelihood of instability of excavations, particularly under the influence of groundwater and surface water from heavy rains seeping down to the subsoils. During periods of heavy rains, stagnant pools of water were observed to seep into the subsoils at slow rates due to the significant clay content in the subsoils.

Soil-Water Chemistry and Corrosion Potential

There is insufficient information on the water quality and soil chemistry at the site. But since the land was used for agricultural purposes in previous years, the presence of potentially active chemical residues from fertilizers and pesticides in the soil and the groundwater is possible. The clay pipe is inert to most chemicals. However, couplings, if metal or rubber, may be susceptible to damage by chemicals in the groundwater and subsoils. Photographs of couplings do not indicate any significant damage of the couplings that may be attributed to the chemicals in ground water and the soils. Any corrosion of metal couplings and fittings may have been initiated by structural distress. When such couplings or fittings are overstressed, microcracks form, which tend to react to moisture and solutes in the soil and begin to deteriorate by corrosion.

Review of Inspection Records

The authors reviewed the inspection records made available by the City to assess the following:

- Contractor's competence and experience on deep sewer construction
- Contractor's and Inspector's understanding of the interdependence between sewer design, installation and eventual performance
- Contractor's quality of work
- Inspector's competence to supervise deep sewer construction
- Conformance/ non-conformance of final installation conditions with the initial design of pipes

No records were made available by the contractor on how the sewer system was constructed, whether any ground water was encountered and when ground water was encountered how it was handled, and if there were any unstable trenches during the initial installation. The Inspectors on several occasions observed and brought to the attention of the contractor's representative poor compaction of backfill and soft spots. The inspectors' daily reports and field notes document the above and other instances in detail. These also indicate that the field installation by the contractor was not generally adequate, particularly lack of adequate compaction of bedding and backfill in pipe trenches. No records are available on the actual condition of the trenches prior to laying of the pipe and how the pipe embedment zone was installed. However, an entry by one of the inspector provides some information on how the deep sewers may have been laid. The entry indicates the site supervision personnel were not aware of the requirement for good bedding for deep VCP installation, and probably did not inspect, monitor, or require the contractor to provide good pipe bedding during the installation of deep VCP sewer mains and laterals. The inspectors were thus negligent of their duty to the city. However this does not absolve the contractor of his responsibilities and the contractor is still liable as he failed to comply with the requirements of the specifications. A chronological summary of notes and comments prepared by the authors based on all the field logs and inspections notes indicated that it was questionable if the VCP installation was completed per specifications. Also, most inspectors assigned by the two firms on contract with the City were not competent, trained or experienced to inspect or monitor deep sewer construction.

Review of Photographs, Videos, and Other Logs

Video recording of pipelines show internal leaks and pipe breaks at a particular point of time. Normally on a sewer project, if the pipe has yet to be commissioned into service, such would represent deficiencies of materials and workmanship by the contractor. Typical video inspections of sewers may miss other imminent sewer failures that may occur at a later time. Usually later inspection will show greater deterioration, pipe breaks and leaks if no repairs were done after the earlier inspection. Video inspection normally provides locations where damage has already occurred or are likely to occur. It is useful in locating pipe and joint failures, but not necessarily the cause of pipe and joint failure.

Most of the repairs were undertaken during the rainy winter months with excessive groundwater. Photographs of excavations, trench collapse, damaged pipes and couplings also clearly show the initial construction to be inferior. This is one of the main reasons why subsequent repairs of previous failures, using great care, are still springing leaks or failing.

Review of Project Specifications

The project contract specification while providing flexibility placed most of the responsibility on the contractor. It is not clear whether the contractor had sufficient experience in handling a contract of this nature, or the contractor was able to provide sufficient supervision of sub-contractors who were less experienced in carrying out the work. There are no records from the contractor or the City regarding the contractor's past experience on similar projects, to evaluate any of the above. However, the performance of the contractor on this project and the field records of the site supervision firms raise doubts regarding the capability of the contractor to install deep sewers. The contractor did not submit a change of site conditions order during the execution of the project. The authors understood this to mean that the contractor did not encounter conditions contrary to the site investigation report.

Bedding/Backfill Materials requirements can be considered to be marginally adequate for the installation of VCP deep sewers and laterals (except the risers) on this project and is generally consistent with similar projects. The depth and trench width requirements are clearly stated on the City details and repeated in the project specifications, and on project drawing sheets. City details for the sewer service lateral installation were also made available as part of the contract.

Lack of Conformance to Contract Specifications

Given the wet weather during construction and the site conditions, the contractor would have required methods, materials, and workmanship of the highest quality to install the deep sewers to the design requirements, contract specifications, and project schedule. The construction procedures did not conform to the requirements of the project specifications. No change orders were authorized by the city or the Engineer or the Design Consultant for any changes from the original design details. As per Section A-20 of the specifications, any changes made to the design by others without prior notification absolves the Design Consultant of responsibility. There is no record of who, when, why, and how changes were made to allow the as-built details and methods. The non-conformance of the as-built conditions from the original contract specifications and as well as the engineering evaluations are as follows:

Trench Width: The original specifications called for minimum trench width of O.D.+ 60 cm (24 in). This was changed to vary from O.D.+ 60 cm (24 in) min. to O.D.+ 120 cm (48 in) max. depending on the shield dimensions. This change impacts on the original assumptions and design of the VCP deep sewer. The earth load increases significantly with increase in trench width.

Pipe Bedding: This was changed from clean sand to 20 cm (8 in) thick 18 mm (¾ in) crushed aggregate. This was a beneficial change, but was negated by the method used for the installation of the pipe embedment zone A.

Zone "A" Pipe Embedment: Unwashed sand, very similar to the well graded clean sand in the original contract specifications was selected. However, moisture consolidation was allowed for the installation of the pipe embedment. This was a very poor choice for compacting the embedment to ensure a proper pipe-soil system. When the unwashed sand is moisture settled over a porous 18 mm (¾ in) bedding not only the fines but even a good portion of the sand would wash away. A geofabric should have been used to prevent such migration of fines. The geotechnical consultant provided recommendation for the use of geofabrics only after the first series of failures.

Backfill Zone "B" First Lift: No compaction requirements are shown on the details. This was probably dumped during deep sewer installation. The requirements vary significantly from the original contract specifications.

Relative Density: In the detail reference is made to "compact to 90% relative density" for the zone "B" and zone "C" backfills in the trench detail. The use of the term "relative density" by the originator/preparer of this figure is considered to be a typo for "dry density" by the authors. On the other hand, it could have been due to lack of understanding of principles of soil engineering by the design engineer of record for this trench detail.

Relative density (for sandy soils) and dry density are not the same and are very different. It is normally not possible to achieve compaction to 90% relative density in sands, but 90% and even 100% dry density is possible Also note the term "relative density" is applicable only to cohesionless soils like sands.

Concrete or Compacted Bedding Requirement for Risers: The National Clay Pipe Institute (NCPI) provides publications on design and installation as aids to the designer and contractor. The requirements for riser installation from the NCPI Clay Pipe Installation Handbook (1994) are shown in their design manual. These guidelines represent minimum requirements for the safe installation for the long term satisfactory performance of the sewers. The importance of compaction of bedding material under the riser bends is also clearly highlighted in these details. In principle, the contractor performed the compaction as per these details, but lacking specific documentation from the site supervision firm, it is not known whether the contractor performed such compaction.

Deep Sewer Lateral Service Connections: The city's details are only good for service connection for shallow sewers. For deep sewers service connections the risers and bends should have been encased in concrete. Typical details of less important pipe installations than the deep riser installation on this project but requiring concrete/crushed stone encasements are shown in the design drawings, which might have misdirected the installer. This was clearly an oversight and possibly negligence, by the Design Consultant, who prepared the original contract designs, drawings and specifications. The details shown for the vertical risers are clearly not adequate for the type of soil conditions, and the fast construction speed required on this project. The project contract specifications and as-built conditions as reported by the design firm and the site supervision firm were compared against the minimum requirements that should have been provided on this project. Ideally the vertical risers must have been encased in concrete. Ensuring adequate compaction is normally difficult even for engineered fills mainly because of space limitations.

Review of Structural Design and Installation of Vitrified Clay Pipe

Clay pipes have been used for hundreds of years in North America and even longer in other countries. For sanitary applications in projects of similar size as this project VCP was correctly considered as one of the suitable alternatives and selected by the designer and the City. Pipe installation refers to the installation of the pipe-soil system which consists of the pipe and the surrounding soil. Pipe manufacturers who recognize the importance of this provide to the installer strict guidance on pipe-soil system installation procedures. For clay pipes in addition to the manufacturer's guidance, such information could have been easily supplemented by the NCPI Clay Pipe Engineering Manual (1995) and Installation Handbook (1994). Pipe system design, installation and performance are inter-dependent. Depending on the type embedment the design load on the clay pipe can vary widely. If the clay pipe is encased in concrete the load carrying capacity of the clay pipe can be almost three times than the clay pipe in the embedment specified for this project, even though the same clay pipe is used in both cases. If the bedding and embedment provided on this project was not up to the requirement of the contract specifications, the load carrying capacity would be reduced. Thus if decisions were made in the field regarding installation procedures or changes in bedding, foundation, embedment or backfill materials, these would impact the design loads and other initial design assumptions and must have been referred to the original design consultant for approval. The recognition of the inter-dependence of VCP design and installation by the designer, contractor and the inspector is very important for the satisfactory long-term performance of clay pipes. Added to this are the significant differences between shallow and deep sewer and increasing complexities of the design, installation, inspection and monitoring.

The Design Consultant's input was limited to almost none during construction of this project. The review of the structural design thus includes the assessment of whether the original pipe design would be adequate for the actual as-built conditions as well as the originally intended installation conditions. It is the conclusion of the authors that the original pipe design and original installation specifications were adequate for the project, except for the deep riser detail and encasement requirements for the service connections. The above conclusion is based on the assumption that a good contractor well experienced in deep sewer construction would install the deep sewers. From a constructability point of view, installing the deep VCP sewers in conformance with the contract drawings, specifications and schedule would have been very difficult particularly for a normal contractor given the possible poor ground behavior under wet conditions which persisted during the period of construction. The Design Consultant should have recognized possible problems of installation and come up with better design details, or better specifications. Furthermore, the Design Consultant should have followed established design guidelines prevalent in the industry for deep sewers and evaluated the suitability of the city details for the deep sewer applications, and provided proper alternative riser and lateral details.

Installed conditions

A good contractor would have at least referred to the NCPI's Clay Pipe Installation Handbook (1994) and have been aware of the importance of the trench width on the load carried by the pipe. The geotechnical engineering report and the specifications indicated to the contractor clearly the poor site conditions. Nevertheless, based on the inspection notes, the contractor ended up with wider trenches than shown on the contract drawings and specifications. If an adequate shoring had been provided by the contractor some of the collapse which led to wider trench conditions could have been avoided in the first place. It is fair to say that normally a Design Consultant would assume a good contractor would avoid such problems by adhering to the specifications and using common sense. The contractor also installed different types of pipes on the project for storm drains etc,. The installation procedures for each of the pipes were not necessarily the same, as well as the designs. It is apparent the contractor did not fully follow the appropriate procedures and guidelines for the installation of clay pipes on this project.

VCP pipes were laid much deeper on this project than is typical for other pipes on other projects. This increased the likelihood of encountering large volumes of perched groundwater, and loose sands and gravel much higher for VCP mains, risers, and laterals for service connections. This was confirmed during excavations for repairs of broken pipes and leaks as shown on the project photographs. In summary, there are significant deviations from the original contract designs, plans and specifications particularly related to trench width, trench support system, bedding material, bedding compaction criteria, trenching for riser and laterals, pipe jointing, bottom compaction in loose bedding, backfill compaction in main trench, riser trench, pipe haunch zone, and under 1/8 bends, "Wyes" and "Tees", to the as-built conditions.

Installed Conditions Impacting VCP Design

Loads on clay pipes are very sensitive to the trench width, up to the transition width at which point the maximum earth load is attained. This is why ensuring the trench width requirement at the pipe crown per design specifications is important. Ground water softens the bottom and sides of trenches. This results in reduced bearing capacity of the foundation. The design capacity of the pipe is thus limited. Soft bedding spots leads to sagging of the line and causes misalignment and pulling out of the pipe at the joints/couplings. In such cases the pipe and couplings carry a much higher soil load than the design soil load. Proper construction provides a firm and uniform support of the pipe barrel to avoid such problems. Soil migration is caused by the perched ground water and loose silty sands in the close vicinity of pipelines particularly with porous bedding/embedment conditions. Soil migration causes sagging of pipelines initially. This possibility existed for the VCP deep sewer mains and the horizontal stretches of the VCP laterals for service connections. Sand is unsuitable as a bedding material if high and rapidly changing water tables are present in the pipe zone. This is also noted in the Clay Pipe Engineering Manual published by NCPI (1995). This type of conditions also existed on the project and such conditions were encountered when excavations were performed for repairs of leaks and pipe breaks. Lack of proper and adequate compaction of hard to get to locations such as under the pipe haunch and under pipe risers leads to subsequent problems. During excavations of repairs of the risers and laterals soft spots were observed and recorded by the inspectors indicating this specification requirement was not conformed to by the contractor. The use of performance specifications for compaction of backfill and limiting compaction tests to the top 1.6 m (5 ft) meant the deeper layers were not well compacted. There were also no proper criteria on what native select fill meant with regards to both soil content and moisture content. Poor installation of bedding, embedment, and backfill lead to pipe breaks and pulling out of joints along riser sections of service connections. Furthermore, after such initial failure of risers, following periods of rain conditions deteriorate rapidly when water seeps into the ground gradually undermining the bedding under horizontal stretches of pipes by carrying the soil away through the broken vertical riser pipes.

Observations During Repairs

The excavations for repairs at Sta. 76+67 showed sagging of lateral pipes, spigots coming out of bells, broken bells, peeling off of part of the spigots, and significant deflections at couplings. Repairs were done on the basis that fines migration was caused by ground water fluctuations. Accordingly filter fabrics and other geotextiles were used in the repairs. The excavations for repairs at Sta. 71+88 showed no crushed rock bedding under the wye. This indicates poor installation due to the lack of understanding of the importance of the pipe-soil system by the contractor.

Possible Causes of VCP Failures

Vertical risers and service connections in clay pipe installation have historically posed problems, and in most cases leading to failure. Shear failure through backfills have been caused because of poor compaction under the risers and service connections. Causes of CFD #1 VCP deep sewer main and service connection laterals and risers are as follows:

- Poor bedding conditions due to ground water fluctuations and consolidation of embedment.
- Poor compaction of the first lift over the pipe embedment.
- Quality of selected native material used as backfill.
- Trench wall instability, excessive trench widths due to inadequate shoring and support system.
- Changes and deviations from original designs, plans and specifications.
- Poor workmanship by the contractor.
- Lack of understanding of pipe design and installation by the contractor/inspectors.

Deficiencies and Responsibilities

The following section summarizes the project deficiencies due to design, specifications, inspection, monitoring and construction management, based on the discussions in the previous sections. The responsible parties to such deficiencies are also discussed.

Project Design and Specifications

The design consultant and engineer of record of the project are responsible for the following deficiencies in design and specifications:

- Failure to follow accepted deep sewer practices as outlined in ASCE design manual (1982), Clay pipe manufacturers' installation, and engineering design guidelines (NCPI, 1994 and 1995), particularly with regard to vertical risers and laterals.
- Failure to provide proper details for bedding, and embedment of VCP sewers and laterals.
- Failure to evaluate city details which are more suitable for shallow installations for project and site conditions such as poor soil, and groundwater.
- Failure to allow for contractor mistakes and omissions.
- Failure to optimize the design taking into consideration constructability.
- Failure to understand the inter-dependence of design and installation on satisfactory long-term performance of deep sewers.

Inspections and Monitoring

Deficiencies in inspection and monitoring during the installation of the project are as follows:

- Failure to provide competent inspection and monitoring personnel for the project.
- Failure to understand the inter-dependence of design and installation on satisfactory long-term performance of deep sewers.
- Failure to ensure the contractor was providing good bedding and embedment as per design and specifications.
- Failure to record who authorized changes from contract design to those as-built conditions, including allowing different materials, methods such as moisture consolidation during installation.
- Allowing moisture consolidation.
- Limiting compaction tests to top 1.6 m (5 ft) of backfill only.
- Failure to establish moisture content, soil content criteria for native select backfill.

Construction

The contractor entered into a contract with the city to build the project conforming to the contract design and specifications. The contract allowed for change orders. Project construction deficiencies are as follows:

- Failure to provide competent experience in deep sewer construction.
- Failure to understand the inter-dependence of design and installation on satisfactory long-term performance of deep sewers.
- Failure to provide good bedding and embedment as per design and specifications.
- Performing work non-conforming to contract design and specifications without getting proper authorization.

Current Condition of System and Possibility of Future Failures

Given the nature of actual installation, it is very difficult to assess the current condition of the deep VCP sewer system and laterals. The subsoil conditions at the site varied from station to station. There were never distinct horizontal layers of soils of known engineering properties. The soils were always a complex mixture of varying amounts of clay, silt, and sand contents. The pipes were laid in a very inconsistent manner. Installation procedures and conditions changed from location to location. Excavations for repairs of pipe breaks indicated soupy backfill particularly during wet weather in most cases. The higher percentage of clay content of at least the top layers is confirmed by the fact the site became water logged after rains. In sandy soils the water will percolate easily. Thus the actual backfill material at the site varied from saturated silty backfill to wet clayey silt backfill, with the majority of backfill being a combination of the two. Taking into account these variables and uncertainties, the current condition of the deep sewers and laterals were thus evaluated for two soil conditions. The wet conditions were selected for both backfill conditions as they were more critical. It is further assumed the contractor actually provided Class C bedding for burial depths over 4.9 m (16 ft), and Class D bedding for burial depths under 4.9 m (16 ft) at best. However, evaluations were performed for both classes of beddings for comparison purposes. The current condition assessment for the deep sewers and laterals were performed for the following two failure criteria:

- Evaluation of structural capacity of VCP to carry the most probable loads due to the changes in installation conditions (Spangler and Hardy, 1982)
- Evaluation of the foundation against bearing capacity failure using the German ATV methodology (Hornung and Kittel, 1989).

In assessing the conditions earth loads were calculated using the Marston Method in accordance with the Clay Pipe Engineering Manual. No traffic loads were calculated for the deep sewers, as the impact was negligible. For the laterals, two cases were evaluated, one with and the other without traffic load and surcharge loads. Factors of safety against pipe failures and bearing capacity failures were calculated both Class C and Class D bedding systems. For purpose of evaluation a factor of safety of 1.0 is required for the satisfactory long term performance of the deep VCP sewer and laterals. An unacceptable factor of safety against pipe failure indicates the pipe is near its load carrying capacity. When the load carrying capacity is exceeded the pipe will crack and deteriorate. This will lead to unacceptable levels of exfiltration/infiltration, as well as soil migration. An unacceptable factor of safety against bearing failure on the other hand indicates the soil foundation beneath the pipe has failed. This will lead to uneven settlement of pipes, leading to shear failure of pipes, pulling out of pipe joints, pipe misalignment, and eventual pipe breaks resulting in unacceptable levels of exfiltration/infiltration, as well as soil migration.

Summary

Because of the initial poor construction, it is possible there may be other locations where leaks may occur and pipe breaks may occur. Some of these may not occur right away, but may occur within months or years. Unlike other pipes, clay pipe deficiencies are normally detected earlier on in most projects, and once these are rectified in a proper manner, they require minimal long term maintenance. However, the same could not be said of the VCP sewer mains and laterals on this project. Leaks and pipe breaks could occur at any location at any time. The long-term reliability of the sewer system is very low. In addition, it is also now known that the leaks have sprung again at locations where leaks and breaks were detected and repaired. Here is one more pipeline project where the design engineering firm, the contractor, and the site supervision firm simply did not have the competency or prior experience to take on the tasks of designing, constructing, and providing supervision over contractor's workmanship. The city and the rate-paying public will continue to pay a heavy price for others' mistakes.

References

American Society of Civil Engineers (ASCE). (1982) *Gravity Sewer Design and Construction, ASCE Manuals and Reports on Engineering Practice No. 60 (Water Pollution Control Federation Manual of Practice No. FD-5),* ASCE, New York.

National Clay Pipe Institute (NCPI). (1995) *Clay Pipe Engineering Manual,* Lake Geneva, WI.

National Clay Pipe Institute (NCPI). (1994) *Clay Pipe Installation Handbook,* Lake Geneva, WI.

Hornung, K. and Kittel, D. (1989) *Structural Analysis of Buried Pipes*, ATV, Bauverlag GmbH, Berlin.

Spangler, M.G. and Handy, R.L. (1982) *Soil Engineering*, Fourth Edition, Harper and Row, New York.

Cost Comparison of Pipeline Asset Replacement: Open-Cut and Pipe-Bursting

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Abstract

The costs of trenchless projects can vary widely with many factors such as the project size, length, depth, location, surface and subsurface conditions, type of application, etc. Although there have been several studies regarding the cost comparison of trenchless with open-cut methods, practical examples of verifying the reasonableness of actual trenchless construction/replacement costs are still needed. In this paper, the cost of an actual pipe bursting project on the campus of Michigan State University (MSU) is described and compared to estimated costs of several traditional open-cut options. To verify the reasonableness of the MSU pipe bursting cost, a cost estimate based on the quantity of MSU pipe bursting project was prepared. To show the price range of pipe bursting projects, a cost comparison with two other pipe bursting projects is also made. Other project aspects, such as time and social costs are briefly mentioned as additional benefits of pipe bursting method. These analyses indicated that pipe bursting method is much more cost effective and approximately two to three times less costly than open-cut options.

Introduction

Asserting whether a construction project is cost effective requires a clear understanding of all the cost factors associated with the specific conditions of the project. When new technologies and methods are considered as alternative construction methods, due to unknown cost parameters, there is usually hesitation and resistance in accepting the new technology. Therefore, solid comparison studies in costs of trenchless technology and traditional open-cut methods will be helpful in acceptance of these new technologies.

The cost of every pipeline project vary with factors such as the pipe size, depth, length, project, site and subsurface conditions, location, type of application, etc. Considering many parameters influencing the costs, this paper confines some cost

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factors to present a practical example of verifying the reasonableness of an actual trenchless construction/replacement cost.

Information on the MSU Service Road Collapse

A sinkhole appeared just outside of the Power Plant and Laundry Buildings on Service Road on the campus of Michigan State University in June 2004. The main cause of the sinkhole was erosion and structural cracks on the existing clay sanitary sewer pipeline and two manhole structures. The two-foot sinkhole in the manhole structure caused water and sand to be sucked away from the surrounding sanitary structures and created a 25-foot deep hole under the road (Figure 1)

Figure 1 – MSU Service Road Pipe Replacement Construction

Having inspected the pipe under Service Road and the manhole structure with Closed-Circuit Television (CCTV), it was decided to replace nearly 300 feet of deteriorated 18-inch clay pipe. To handle the existing sewage flow properly, an extensive pumping system was used to bypass raw sewage to a downstream manhole structure.

Bidding Process

Before a contractor and a method were selected, bids were obtained from several contractors which included Cured-In-Place-Pipe (CIPP) and Static Pipe Bursting. The following is a list of the bidders and methods obtained:

- Bidder A: 270 ft of sanitary sewer lining, 18 in. diameter by 9 mm thickness of CIPP at $55,350. The contractor at the time did not have enough materials to line the required 310 feet.

- Bidder B: 280 ft of 12 mm thickness of 18 in. CIPP at $59,570 plus $15,840 premium for emergency work.

- Bidder C: 310 ft of 18 in. DR 17 HDPE using pipe bursting method with pit excavation to be done by others at $52,000. This method was selected.

Pipe-Bursting Specifics

For the pipe bursting method, two pits need to be excavated at the entry and the exit of the pipe bursting operation. ASCE Pipe Bursting Manual of Practice No. 112 (ASCE 2007) provides a full description of pipe bursting method and its requirements. With pipe bursting, the entry pit needs a gradual slope to insert the new pipe depending on the type and rigidity of the pipe. For this project, the selected slope for the pipe entry pit was approximately 1.5 horizontal to 1 vertical. Table 1 presents specifics of pipe bursting operation. Figure 2 illustrates pipe entry and pipe receiving pits.

Cost Analyses for the Open-Cut Construction

Most of the information needed to theoretically calculate the open-cut method for cost comparison purposes is based on OSHA guidelines for excavation safety. The new pipe material assumed with the open-cut method is a Vitrified Clay pipe (18 in.).

Table 1 – Specifics of MSU Service Rd Pipe Bursting Project

- New pipe: DR 17, 18 in. diameter (Iron Pipe Size) HDPE
- Old pipe size: 18 in.
- Total pipe length: 310 ft
- Pit sizes for pipe entry and reception: Approximately 12 ft wide by 25 ft long
- Slope for pit wall on the pipe entry side: 1.5 to 1
- Pipe bursting machine used: TRS 225 ton capacity
- Installation Speed: Approximately between 0.5 to 1 ft/min
- Pipe Bursting Crew: 1 foreman, 1 operator, 3 laborers
- Duration: 2 days (1 day for mobilization, 1 day for the bursting)

Figure 2 – Pipe Bursting Receiving Pit (Left) and Pipe Entry (Right) Pit

According to Figure 3, the following is a list of work items for open-cut method (Howard, 2002):

- Pipe Transportation, Handling, and Storage
- Trench Excavation
- Foundation, Bedding and Laying, and Joining of Pipe
- Embedment, Backfill, and Compaction of the Soil
- Others as necessary

Excavation Options and Quantity Takeoff

To simplify open-cut cost calculations, several options are considered for trench excavation. Also, the ground water table is assumed to be below the bottom of excavation. Thus, dewatering (a major cost item for open-cut) is not considered in the excavation option.

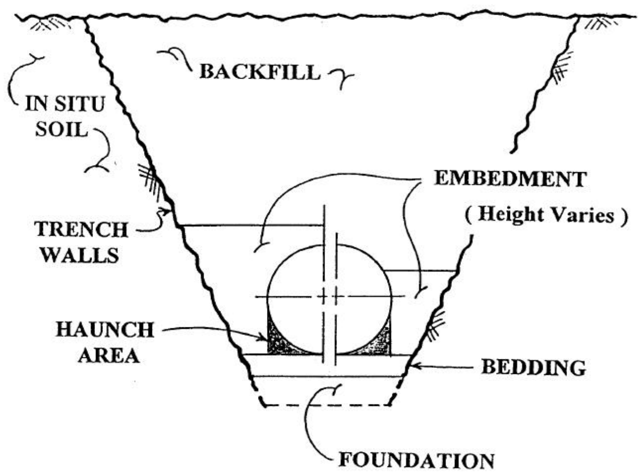

Figure 3 – Open-Cut Pipe Installation Cross Section (Howard, 2002)

Excavations and Protective Systems. Four protective options for open-cut method are considered. Sloping can be expensive and often causes traffic disruption; but it is an option that the contractor can analyze before bidding.

- Option 1. Simple Sloping (Table 2)
- Option 2. Simple Sloping with Benches (See Table 3)
- Option 3. Combination of Sloping and Shielding (See Table 4)
- Option 4. Shoring or Shielding (See Table 5)

For excavations less than 5 feet deep, OSHA generally does not require a protective system unless a competent person at the jobsite observes signs of a potential cave-in. However, it does not mean that no protective systems are required for any excavations less than 5 feet. The options for the excavation presented in this paper follow OSHA's four standard protective systems assuming soil Type B (OSHA, 2006).

Both Options 1 and 2 need 32 feet of surface excavation at the west end and 44 feet at the east end of the project. Considering that the pavement width actually is 33 feet, these two options would require a much larger area to be excavated than the road surface allows. Accordingly, the excavation volume will be very large.

Option 3 utilizes a combination of sloping and shielding system. The shielding on both sides of the bottom 5 feet provides a firm support against the high earth pressure. The sloping 5 feet above the shielding provides enough support to the excavation wall as well as reducing excavation volume compared to sloping only. Option 4 uses shielding on both sides of the trench from the bottom to the top. It requires only 4 feet of surface excavation along the trench, and the excavation volume will be much smaller than Options 1 and 2.

Old Pipe Removal, Bedding, and Laying New Pipe. After 310 feet of the old clay pipeline is removed and cleaned away, the bottom of the trench needs to be prepared for the new pipe installation corresponding to the pipe material. For cost comparisons, same diameter (18-in.) vitrified clay pipe is proposed to be used as a new pipe material.

Since vitrified clay is a rigid pipe, it is assumed that the excavated soil is suitable for the bedding. The bedding thickness is determined as 4 inches which is a general bedding thickness for the pipe of 12 - 54 inches in diameter. Laying pipe involves hoisting the pipe up from the side of the trench and lowering it into place on the trench bottom. The pipe must be laid carefully to avoid damage. Then, the pipe sections will be joined using proper procedures.

Backfilling and Compaction. After the pipe is laid, embedding material will be placed around the pipe. The embedment and the pipe act together as the pipe-soil structure to support the external loads on the pipe. Similarly to the bedding material, it is assumed that the excavated soil will be used for embedment. Then, the same soil will be used again for the backfilling.

Pavement Removal and Replacement. The existing asphalt pavement needs to be removed before excavation. After the new pipe is installed and the trench is backfilled and compacted, the road will be paved with bituminous material.

Selected Options and Quantity Takeoff. Tables 2, 3, 4, and 5 present quantity takeoffs for the construction operations using open-cut options. The estimates include the amounts of excavation, pavement construction, shielding, mobilization, pipe bedding, embedding, and backfilling. Since Option 1 (simple slope) and Option 2 (slope with benching) require large areas for excavation and pavement replacement as well as large amounts of excavation volume, only Options 3 and 4 were chosen for the cost estimate.

Table 2 -- Excavation Quantity Takeoff for Option 1: Simple Sloping

Length =		310	ft						
Depth =		14	ft			Depth =		20	ft
Top =		32	ft			Top =		44	ft
Bottom =		4	ft			Bottom =		4	ft
Area West:	(Depth) * (Top+Bottom)/2 =					252	sf		
Area East:	(Depth) * (Top+Bottom)/2 =					480	sf		
Volume =	(Area W + Area E)/2 *Length =					113,460	CF		
					=	**4,202**	CY		

Table 3 -- Excavation Quantity Takeoff for Option 2: Sloping with Benching

Length =		310	ft								
Depth =		14	ft					Depth =		20	ft
D 1 =		4	ft					D 1=		4	ft
Db =		2.5	ft					Db =		4	ft
Top =		32	ft					Top =		44	ft
Bottom =		4	ft					Bottom =		4	ft
Area West:	(Depth)*(Top+Bottom)/2 - 8*(0.5*2.5*2.5) - 2*(0.5*4*4) =									211	sf
Area East:	(Depth)*(Top+Bottom)/2 - 10*(0.5*4*4) =									400	sf
Volume =	(Area W + Area E)/2 *Length						=	94,705	CF		
							=	**3,508**	CY		

Open-Cut Cost Estimate

Based on the quantity takeoff presented above, the cost for the open-cut method (Options 3 and 4) has been estimated. The construction cost items include: mobilization, pavement removal and replacement, excavation, shielding, old pipe removal, pipe bedding, installing new clay pipe, embedding, backfilling, compaction, and demobilization. General or indirect cost items include contingency, overhead and profit, bond, and insurance. The unit cost data of each item are mainly from Means Heavy Construction Cost Data 2005. The total estimated construction cost is estimated to be **$100,974** for Option 3 (combination of sloping and shielding), and **$137,298** for Option 4. Shielding is the most costly item in both Options 3 and 4. Pavement replacement, clay pipe installation, and excavating the trench are other major cost items in these options (See Tables 7 and 8).

Table 4 -- Excavation Quantity Takeoff for Option 3: Combination of Sloping and Shielding

Length =		310	ft						
Depth =		14	ft			Depth =		20	ft
D 1 =		5	ft			D 1 =		5	ft
D 2 =		9	ft			D 2 =		15	ft
Top =		22	ft			Top =		34	ft
Bottom =		4	ft			Bottom =		4	ft
Area West:	(D2) * (Top+Bottom)/2 + (D1*Bottom)=						137	SF	
Area East:	(D2) * (Top+Bottom)/2 + (D1*Bottom)=						305	SF	
Volume =	(Area W + Area E)/2 *Length =					68,510	CF		
						2,537	**CY**		
Shielding =	2 * (D1 * Length) =			3,100	SF				
Mobilize =	Peripheral Length =			688	LF				
Pavement =	33ft * Length =			10,230	SF =	1,137	SY		
Bedding thickness=		4	in						
Pipe Bedding = Thickness*(1/12) ft*Bottom*Length =							413	CF	
						=	15.3	CY	
Volume for Pipe =		548	SF	=	20	CY			
Embedment & Backfill = Excavation - Bedding - Pipe volume =								2,502	CY

Table 5 Excavation Quantity Takeoff for Option 4: Shoring or Shielding

Length =		310	ft							
Depth =		14	ft				Depth =		20	ft
Width =		4	ft				Width =		4	ft
Area West:	Depth * Width =						56	sf		
Area East:	Depth * Width =						80	sf		
Volume =	(Area W + Area E)/2 *Length =						21,080	CF		
							781	**CY**		
Shielding =	2 * (Depth West + Depth East)/2 * Length =								10,540	SF
Mobilize =	Peripheral Length =			628	LF					
Pavement =	6ft * Length =			1,550	SF	=	172	SY		
	(4ft + 1ft each side)									
Bedding thickness=		4	in							
Pipe Bedding =		Thickness *(1/12) ft * Width * Length =						413	CF	
							=	15.3	CY	
Volume for Pipe =		548	SF	=	20	CY				
Embedment & Backfill = Excavation - Bedding - Pipe Volume =									745	CY

The major cost difference between Options 3 and 4 is due to the amount of shielding and pavement replacement used. Shielding has several advantages in that it provides a firm support to the trench walls, and it reduces surface disruption. However, it can increase the total cost greatly if shielding is used all along the trench wall. By using shielding materials only to the bottom of trench (Option 3), the shielding costs can be decreased. As shown in Tables 7 and 8, the costs of pavement replacement and volume of excavation-backfilling also attribute greatly to the total cost. When using less shielding material (Option 3), the cost of pavement replacement and excavation-backfilling increases.

Among all the open-cut options included in these analyses, Option 3 (combination of sloping and shielding) is chosen for comparison with the pipe bursting method. Option 3 has relatively small effects to the environment and is less expensive than Option 4 while satisfies the safety requirements. The cost estimates for open-cut Options 3 and 4 are summarized in Tables 7 and 8 respectively.

Pipe Bursting Cost Estimate

The theoretical cost estimate of pipe replacement construction using pipe bursting method should include the following general work items:

- Bypassing
- Pre-inspection
- Mobilization of equipment
- Excavation of access pits (including pavement removal)
- Removal of existing cleanouts
- Pipe fusing
- Pipe bursting operation
- Closing of pits and surface restoration (including pavement replacement)
- Demobilization of equipment
- Final Inspection
- Construction of manholes

However, for simplicity, this paper considers mainly, the following work items: initial inspection; pipe replacement (mobilization/demobilization, pipe bursting operation, pit excavation, pavement removal and replacement); and final inspection.

Table 6 presents quantities for the pipe bursting operation. Two pits (a pipe insertion pit and a pipe reception pit) needed at each end of pipe run. The pit sizes are based on the actual pipe bursting implemented on MSU Service Road (25 ft long * 12 ft wide * 14 ft deep for entry and 25 ft long * 12 ft wide * 20 ft deep for reception). The pavement is removed and replaced for the pit excavation only. Because the bypassing work and the manhole reconstruction were conducted by all other options, it is not included in the cost comparison.

The total *estimated* cost of pipe bursting method is **$71,965** while the pipe bursting portion of the cost is **$57,970** as shown in Table 7. Compared to the open-cut options, the pipe bursting method helps reduce the amount of excavation, backfilling, compaction, and pavement replacement significantly. The most notable cost factor in the pipe bursting option is the pipe bursting operation itself. The lump-sum cost for the pipe bursting portion seems a little high; however, the cost is significantly less than the open-cut method. Additionally, if the length of pipe replacement increases, the cost difference between the pipe bursting and open-cut methods would be even greater. This is because the pipe bursting portion of the operation, which is the most costly item in the pipe bursting method, is estimated on a linear foot basis while excavation-backfilling, pavement replacement, and shielding, which are the major cost items in open-cut method, are estimated on a cubic yard or square foot basis. In addition to the construction cost, the costs due to the environmental and traffic disruption are reduced when pipe bursting method is used since it occupies much less surface area.

Table 6 -- Pipe Bursting Quantities

Work Item	Calculation				Qty	Unit
Pre-Inspection		310	LF	=	310	LF
Mob/Demob		1	ea	=	1.00	Job
Pavement Removal	= (25 ft*12 ft)*2 =	600	Sq ft	=	66.67	Sq yd
Entry Pit Excavation	= (25 ft*12 ft*14 ft) =	4200	CF	=	155.56	CY
Exit Pit Excavation	= (25 ft*12 ft*20 ft) =	6000	CF	=	222.22	CY
Pits Backfill & Compaction	= Entry + Exit pits				377.78	CY
Pipe Bursting with 18-in. HDPE Pipe		310	LF	=	310.00	LF
Pavement Replacement	= (25 ft*12 ft)*2 =	600	Sq ft	=	600.00	Sq ft
Final Inspection		310	LF	=	310	LF

Actual Project Costs (Lump Sum Price)

Table 7 presents the actual cost of MSU Service Road Pipe bursting project. This cost information is obtained from the contractor. The total cost of the pipe bursting project was around **$52,000,** where the pipe bursting portion of the operation was about **$30,000** as shown in Table 7.

Table 7 -- Actual Pipe Bursting Project Costs (Based on the Contractor's Bid)

Item	Cost	Description				
Pipe Bursting	**$30,000**	**$96.73**	**/LF**	**for**	**310**	**ft**
	$22,000	Other costs (including additional excavation, asphalt pavement, indirect costs, etc.)				
Total Cost =	**$52,000**					
Total cost per linear foot =		**$168.35**	**/LF**	**for**	**310**	**ft**

Cost Comparison of Open-Cut and Pipe-Bursting Methods

The cost for the MSU Service Road pipeline replacement using open-cut method using Option 3 (Combination of Sloping and Shielding) was estimated as $100,973 **($325.72/LF)**. The cost for Option 4 (Shoring or Shielding) was estimated as $137,297 **($442.90/LF)**. In these open-cut cost estimates, the cost varied mainly depending on the excavation wall protective systems. These costs are much higher than the *estimated* cost for pipe bursting ($71,965 or **$232.15/LF**), and the actual pipe bursting construction cost ($52,187 or **$168.35/LF**). Table 8 presents the overall cost comparisons.

If the length of the pipe replacement increases, then the cost per linear foot for open-cut method will increase; however, the cost per linear foot for pipe bursting method will not change significantly. This is because the open-cut method requires excavating the trench all along the pipeline with a corresponding protective system installed, but pipe bursting will still have the same excavation amount of entry and exit pits within acceptable ranges. To show effects of pipe size and length (assuming all other parameters are same), several other project costs were compared with MSU project. For example, the Sunny Meadows pipe bursting construction cost shows a project length of 2,637 feet which is almost nine times longer than the MSU Service Road. However, the cost per linear foot was **$159.13/LF** which is slightly less than MSU Service Road pipe replacement. Pipe size another factor that determines pipe bursting cost. If the pipe size is fairly small, then the cost per linear foot will be significantly less as it can be seen in the West Vancouver case **($65.65/LF)**. The MSU Service Road (2004), Sunny Meadows (2000), and West Vancouver (2003) cases were in different project and site conditions; however, they can *indicate* how cost can vary depending on the pipe length and the pipe size and/or amount of upsizing.

Table 8 -- Cost Comparison of Open-Cut vs. Pipe Bursting

	Project	Total Replacement Cost	Total Cost per linear foot		Project Length		Pipe Size		New Pipe
MSU Service Rd	MSU Service Rd Open-Cut Option 4	**$137,297**	**$442.90**	**/LF**	**310**	ft	18	in	HDPE
	MSU Service Rd Open-Cut Option 3	**$100,973**	**$325.72**	**/LF**	**310**	ft	18	in	HDPE
	MSU Service Rd *Estimated* Pipe Bursting	**$71,965**	**$232.15**	**/LF**	**310**	ft	18	in	HDPE
	MSU Service Rd *Actual* Pipe Bursting	**$52,187**	**$168.35**	**/LF**	**310**	ft	18	in	HDPE
Others	Sunny Meadows Pipe bursting	($419,617)	**$159.13**	**/LF**	**2637**	ft	16	in	HDPE
	West Vancouver Pipe bursting	($78,346)	**$65.65**	**/LF**	**907**	ft	4	in	HDPE

Pipe replacement cost will vary with numerous different parameters such as the pipe size, pipe length, frequency of connections, and project surface, subsurface and other characteristics. Thus, while cost of a project is unique and cannot be compared with other projects, cost comparisons can be made for reality checks as long as there are consistencies among major project variables. More research is needed to correlate pipe bursting costs for different project parameters.

Conclusions

These cost analyses in this paper indicated that pipe bursting method is much more cost effective and approximately two to three times less costly than open-cut options. The actual MSU Service Road cost was lower than the estimated pipe bursting cost from the Means Cost Data. It was also shown that the actual cost of MSU Service Road pipe bursting project was within a reasonable range of the average pipe bursting costs in two other pipe bursting projects.

References

ASCE (2007). "Pipe Bursting Projects," ASCE Manuals and Reports on Engineering Practice No. 112, American Society of Civil Engineers, Reston, Virginia.

Gokhale, S., Short, T., and Stockamp, C. (2000). *Pipe bursting case study: Sunny Meadows sanitary sewer rehabilitation. No-Dig Engineering*, VOL. 7 (No.2). Retrieved October 2004 from **http://www.nodigengineering.com/ndesample.pdf**

Howard, A. (2002). *Pipeline installation: A manual for construction of buried pipe*, Relativity Publishing, Lakewood, CO.

Kyes, W.S. Specialist, Mechanical Service, 129 Physical Plant, East Lansing, MI, Fall 2004

Longoria, R. (2004). *Service road develops a sinkhole*, Michigan State University Physical Plant Newsletter Summer 2004, Retrieved December 2004 from **http://www.pp.msu.edu/newsltrs/Summer%202004.pdf**

Michigan State University (MSU) Physical Plant, East Lansing, MI, 2004~2005

Najafi, M. and Gokhale, S. (2005). *Trenchless technology: Pipeline and utility design, construction, and renewal*, McGraw-HILL, New York, NY.

Occupational Safety & Health Administration (OSHA). (2006). Excavations: Hazard recognition in trenching and shoring, OSHA Technical Manual, Visited May 2006 from **http://www.osha.gov/dts/osta/otm/otm_v/otm_v_2.html**

Prystai, T., Sangha, G., and O'Sullivan, D. (2003). *Case study – West Vancouver (pipe bursting)*, District of West Vancouver and Dayton & Knight Ltd. (D&K), Retrieved October 2004 from **http://www.latech.edu/tech/engr/ttc/werf/6.case%20study.west%20vancouver.pdf**

Spencer, E.R, Balboni B., Bastoni, R.A, *et al.* (2004). *RSMeans heavy construction cost data 19th annual edition (2005)*, RSMeans Company, Inc., Kingston, MA.

Suprenant, B.A. and Basham, K.D. (1993). *Excavation safety: Understanding and complying with OSHA standards*, The Aberdeen Group, Addison, IL.

Rising Water and Wastewater Pipeline Construction Costs: A Survey of the DFW Metroplex Marketplace

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Background

Water and wastewater pipeline construction costs have consistently increased over the years. However, pipeline construction costs recently have seen significant increases. Because demand in rehabilitation of existing pipelines and construction of new pipelines continues to grow, design engineers face a challenging task when asked to prepare an opinion of probable construction costs.

The purpose of this paper is to identify trends and/or major differences within the Dallas/Fort Worth (DFW) metroplex, as well as discuss some of the factors influencing the increased costs of water and wastewater pipeline construction.

We collected bid tabulation data from the past five years for 8- to 18-inch pipelines from 17 cities around the DFW metroplex. This data was used to compare the cost of pipeline construction based on size of pipeline, type of installation, and population of municipality. The type of installation examined included open cut and trenchless technology methods.

Construction Cost Trends by Method of Construction

Open Cut

The bulk of the bid tabulation data collected was for open cut construction, including 60 bids with total construction costs ranging from $70,000 to $7,500,000. **Figure 1** illustrates the average of the low bid costs per linear foot for each pipe size on a yearly basis. Immediately noticeable is a sharp increase in costs between 2000 and 2002, which is consistent for almost all pipe sizes. The increases between 2000 and 2002 are followed by a consistent decline between 2003 and 2005, and a very sharp increase in 2006. These cost ranges are presented in terms of percentage changes by year for each pipe size in **Table 1**.

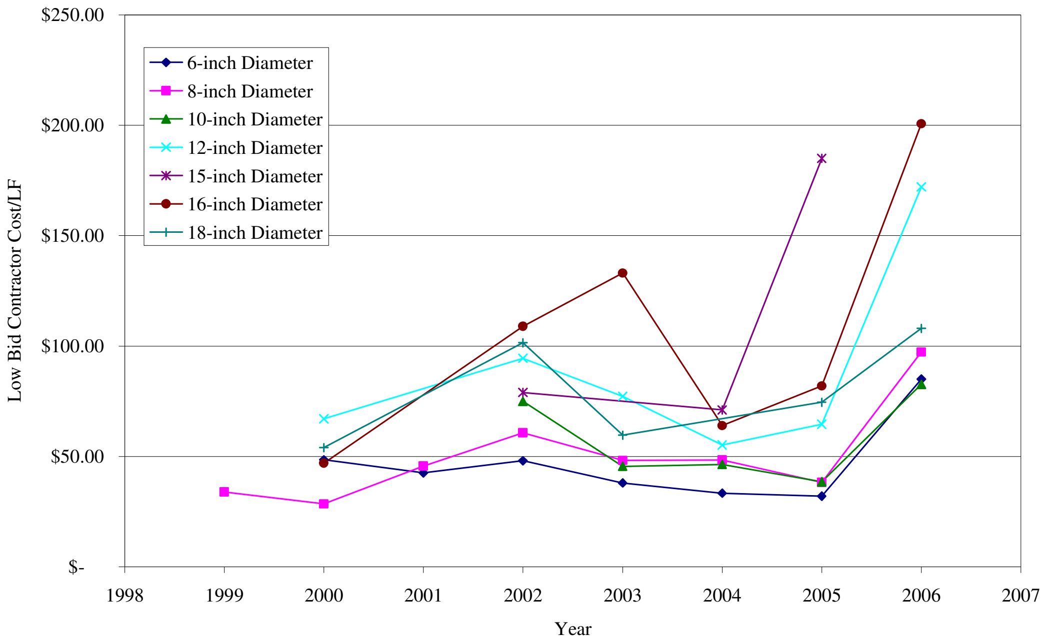

Figure 1 – Open Cut Cost Comparison

Table 1 - Percent Change per Year (2000-2006)

	Yearly % Increase						Average yearly % Increase
	2000-2001	2001-2002	2002-2003	2003-2004	2004-2005	2005-2006	
6	-12.37%	12.94%	-20.83%	-12.10%	-4.16%	165.40%	21.48%
8	60.21%	32.75%	-20.51%	0.33%	-20.86%	153.98%	34.32%
10	-	-	-39.33%	1.98%	-16.88%	114.17%	14.99%
12	-	20.52%	-18.25%	-28.61%	17.18%	166.25%	31.42%
15	-	-	-	-5.06%	160.56%	-	77.75%
16	-	65.96%	22.02%	-51.88%	27.97%	145.00%	41.81%
18	-	43.98%	-41.26%	-	12.52%	68.75%	21.00%

Trenchless Technology Methods

Trenchless technology construction methods include boring, horizontal directional drilling, cured-in-place pipe, and pipe bursting. The information tabulated includes 37 trenchless technology bids with total construction costs also ranging from $70,000 to $7,500,000. **Figure 2** illustrates the average of the low bid costs per linear foot for each pipe size on a yearly basis. The trends for the trenchless technology methods of construction are similar to those for open cut construction, with limited differences between 2002 and 2003, and 2004 and 2005. **Figure 3** compares the 8-inch open cut costs to the costs for 8-inch by trenchless technology methods. From 2002 to 2003, the open cut costs show a decrease of approximately $10/LF (about 16% of construction costs) while the trenchless technology methods trend is an increase of approximately $30/LF (about 20% of construction costs). The open cut costs also decreased between 2004 and 2005, while the trenchless technology methods costs had a significant increase.

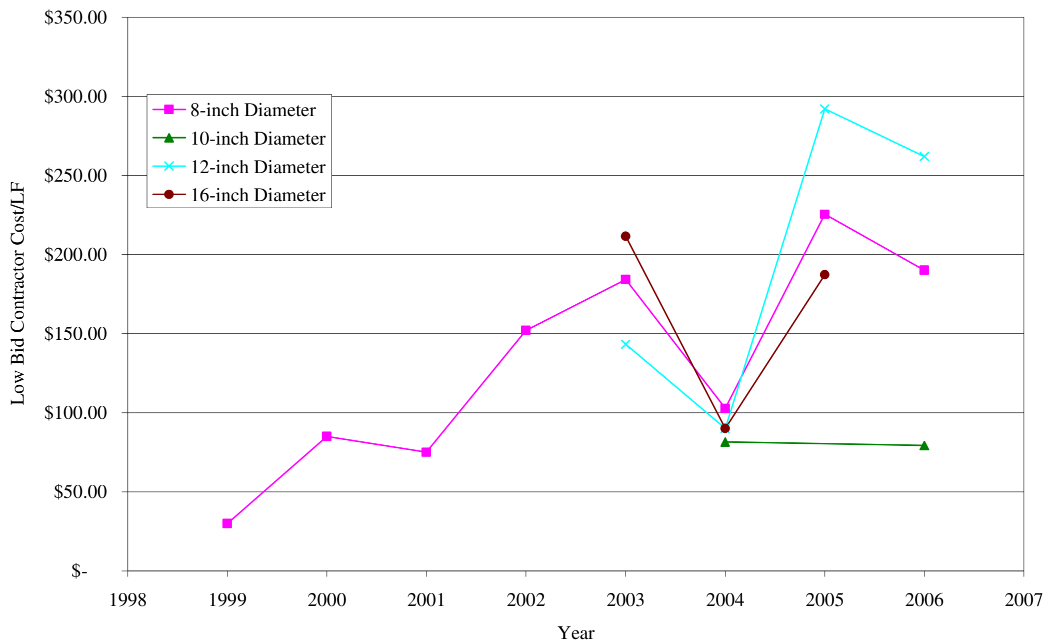

Figure 2 – Trenchless Technology Methods Comparison

Figure 3 – Trenchless Technology Methods Cost compared to Open Cut

Cured-in-Place Pipe
Table 2 illustrates the cured-in-place pipe construction costs ranging from $44.00 per linear foot to $100.00 per linear foot between 2002 and 2005.

Table 2 – Cured-in-Place Pipe Cost Comparison

	CIPP Unit Price for Low Bid Contractor		
Diameter	2002	2004	2005
6	-	\$44.00	-
8	\$100.00	\$50.00	\$47.50
10	-	\$81.50	-
12	-	\$90.00	-

Directional Drilling
Table 3 illustrates the directional drilling construction costs ranging from $65.00 per linear foot to $212.00 per linear foot between 1999 and 2005.

Table 3 – Directional Drilling Cost Comparison

	Directional Drilling Unit Price for Low Bid Contractor		
Diameter	1999	2004	2005
8	-	\$146.00	\$65.00
12	-	-	\$95.00
20	\$112.00	-	-
30	\$212.00	-	-

Pipe Bursting
Table 4 illustrates the pipe bursting construction costs ranging from $55.00 per linear foot to $130.00 per linear foot between 2002 and 2006.

Table 4 – Pipe Bursting Cost Comparison

	Pipe Bursting Unit Price for Low Bid Contractor			
Diameter	2002	2004	2005	2006
6	-	\$55.00	-	-
8	\$130.00	\$66.32	\$58.75	\$69.50
10	-		-	\$79.25
16	-	\$90.00	-	-

Construction Cost Trends by City Population

In addition to comparing the types of construction and pipe sizes, the data for municipalities of different sizes was also compared. **Figure 4** illustrates the construction costs per linear foot for 8-inch pipe on a yearly basis for municipalities with a population less than 50,000; a population between 50,000 and 100,000; and a population greater than 100,000. Cities surveyed with a population less than 50,000 include Desoto, Bedford, Colleyville, Duncanville, Coppell, Saginaw, University Park, Cedar Hill, and Fairview. Cities surveyed with a population between 50,000 and 100,000 include Euless, Allen, Richardson, and Flower Mound. Cities surveyed with a population greater than 100,000 include Dallas, Plano, Arlington, Fort Worth, and Grand Prairie. (The data for municipalities between 50,000 and 100,000 is limited to the years between 2003 and 2005.) When comparing municipalities with a population less than 50,000 to municipalities with more than 100,000 residents, it is interesting to note that the trends between 2002 and 2005 are opposite — construction costs for smaller municipalities decreased during the same period that costs increased for larger municipalities. This can be seen most dramatically in the costs between 2003 and 2004. It should also be noted that both the smaller and larger municipalities experience a sharp increase in 2006.

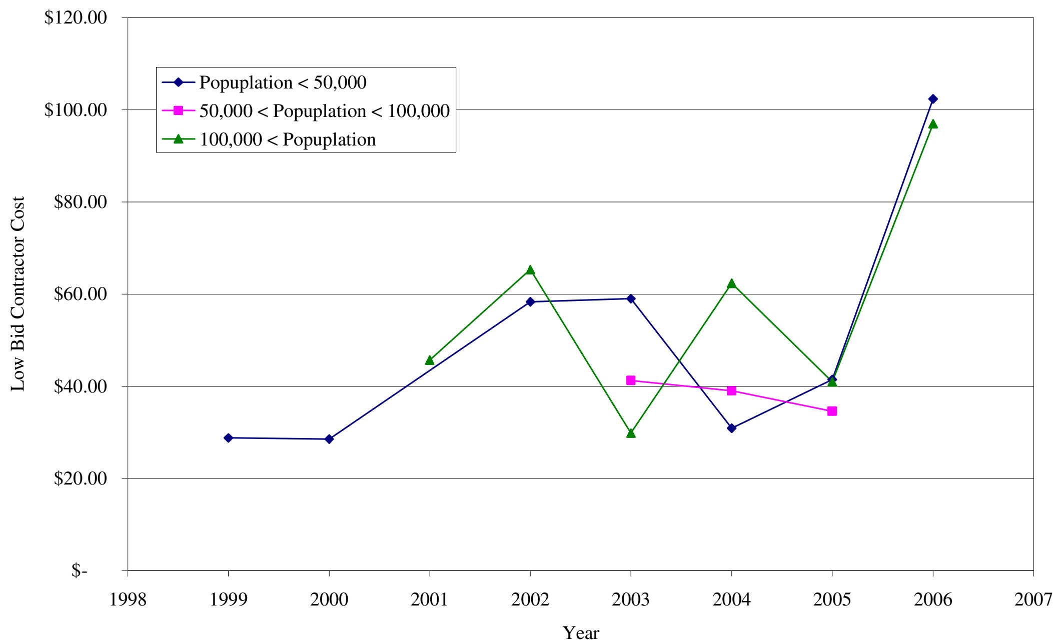

Figure 4 – Population Cost Comparison for an 8-inch Diameter Pipe

Survey with Municipalities and Pipe Manufacturers

After bid data was received, we investigated the cost of pipe within the metroplex by interviewing municipalities and pipe manufacturers to determine if there were specific factors contributing to the rise in pipeline costs. Examples of questions to the Municipalities included:

1) What trends have you seen in your bid tabs for water and wastewater pipeline construction costs over the past five years?
2) Why do you think there has been an increase/decrease in pipeline construction costs?
3) What method of construction for pipe replacement do you prefer? Why?

4) What local requirements or conditions do you have that could impact the pipeline construction costs when comparing to other municipalities?

Municipalities asked to participate in our interviews included the Town of Flower Mound, the City of Dallas Water Utilities department, and the Cities of Allen and Arlington. All of the municipalities saw an increase in pipe costs within the last couple of years. One of the municipalities responded, "the costs of construction have increased as much as 25%. In the recent year, we have had to re-bid projects due to budgetary constraints." In response to our question as to why city staff thinks costs have increased, the answers were similar — the cause in rising prices are due to limited resources, lack of materials, rising fuel costs, and increased cost of materials. Also, due to the surplus of construction projects throughout the DFW metroplex (and north Texas), contractors are being more selective on their bids, making them less competitive.

In addition to interviewing municipalities, we also contacted pipe manufacturers and representatives to inquire about their observations regarding construction costs. Sample questions asked to the pipe manufacturers included:

1) What trends have you seen in the cost of pipe over the past five years?
2) Are there any materials that have increased/decreased more than others?
3) Why do you think there has been an increase/decrease in cost of pipe?
4) What do you predict for the future cost of pipe in the next couple years?

Manufactures who responded included American Cast Iron Pipe, US Pipe, and Hanson. American Cast Iron Pipe mentioned that the cost of ductile iron piping has increased over 40-50% within the last four years. This increased cost was attributed to the increase in cost for scrap steel, which is the primary raw material used in the production of ductile iron pipe. The high demand for scrap steel in China is a key contributor to the increase in cost for scrap steel. In addition to the cost of scrap steel, the costs of energy and natural gas have skyrocketed, as have transportation and fuel costs. Hanson also noted the increase in pipe costs. After the hurricanes last year, the supply of PVC was limited due to resin manufacturers unable to transport from the manufacturing plants in the Gulf of Mexico to pipe manufacturers. This unavailability caused a spike in PVC pipe costs.

Explanation of Findings

The factors influencing water and wastewater pipeline construction costs can be divided into three categories:

1) Local Conditions
2) Market Conditions
3) External Conditions

Each of these conditions can influence the pipeline construction costs to go higher or lower depending on the specific factors involved.

Local Conditions

Local conditions include elements such as project length, depth, adjacent and crossing utilities, service connections, pavement replacement, traffic control, backfill, trench excavation material, municipal bureaucracy, and disadvantaged and minority-owned business participation goals. Local conditions can vary significantly from project to project within the same calendar year. Longer project lengths, shallower depths, fewer crossing utilities, less pavement replacement, and native backfill material will result in lower unit costs for construction. However, municipalities implementing strict procedures and disadvantaged and minority-owned participation goals that are difficult to achieve can expect higher pipeline construction costs.

Market Conditions

Market conditions include elements such as the cost of materials, cost of petroleum, labor costs, workforce supply, and the overall U.S. economy. Each of these factors will not vary as much as the local conditions from project to project within the same calendar year. However, when market conditions change, the impact on pipeline construction costs can be significant, especially when two or more factors move in the same direction.

External Conditions

External conditions include things like natural disasters, terrorist activity, or political actions. These conditions are outside of the actual project and pipeline market, but can have a major impact on pipeline construction costs. Two specific examples substantiated by the bid data we analyzed are the terrorist attacks of September 11, 2001, and the hurricanes on the Gulf Coast in 2005. Bids in 2002 and 2006 were significantly higher than bids in 2001 and 2005, respectively. The terrorist attacks of September 11, 2001 influenced market conditions by increasing the cost of petroleum and slowing the overall U.S. economy. The hurricanes on the Gulf Coast in 2005 influenced market conditions by increasing the cost of petroleum (due to reduced supply), the cost of construction materials (due to rebuilding efforts), and labor costs (because of the higher demand for labor forces).

Conclusion

Local and external conditions influence the market conditions to determine the actual water and wastewater pipeline construction costs in the DFW metroplex. The market conditions over the last five years have been heavily influenced by external conditions such as the September 11, 2001 terrorist attacks and the hurricanes of 2005. The result is record-high unit costs in 2006 for water and wastewater pipelines, with several municipalities faced with rejecting bids and/or canceling projects altogether.

When preparing opinions of probable construction costs, most engineers and municipalities have a very good understanding of the local conditions that influence the construction costs. However, without an equally good understanding of the external conditions and market conditions affecting the market, most engineers and municipalities will continue to be surprised by bid results.

Prudent engineers will develop a regular dialogue with pipe manufacturers, suppliers and contractors to understand the market conditions and the external conditions influencing them.

Anticipated Trends

After reviewing the trends over the past few years and gathering input from both municipalities and pipe manufacturers, the expectation for 2007 is that there should not be the same major increase in pipe and construction costs that was observed in 2006. It should also be noted that no one expects a significant decrease in unit costs. At best, unit costs may decrease slightly below the unit costs of 2006 due to lower petroleum costs, but a continued upward trend is expected for pipeline construction costs over the next few years, ranging from 3% to 15% on an annual basis. **Figure 5** is a graphical representation of the anticipated trends for the next few years.

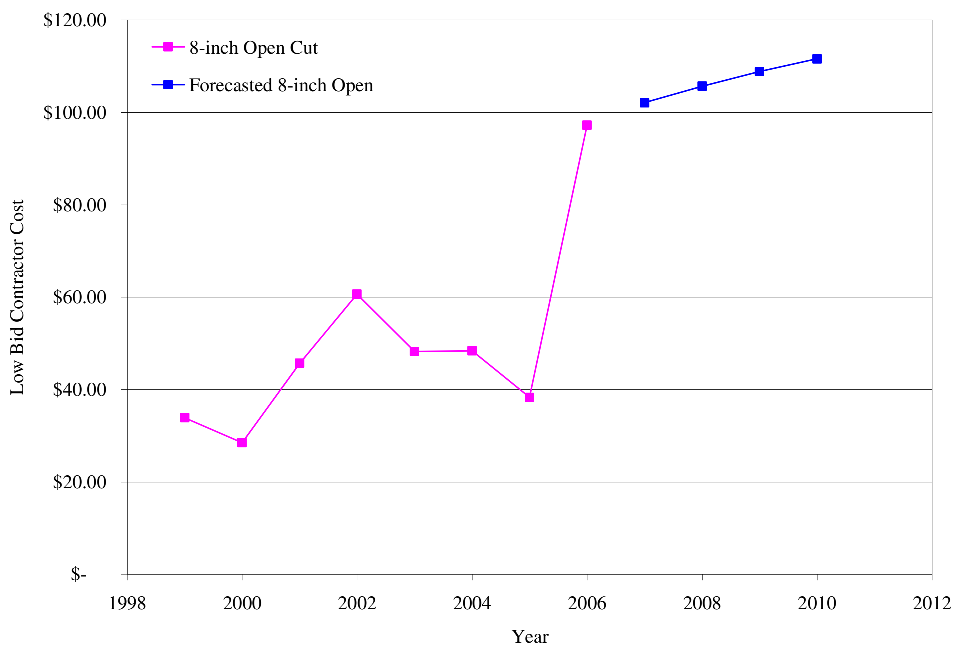

Figure 5 - Anticipated Trends

References

Baptista, Caesar. Email Interview. City of Dallas, Texas. 24 January 2007

Bid Tabs Summary webpage. City of University Park, Texas
www.uptexas.com/index.cfm?FuseAction=page&PageID=000102

Bid Tabulations webpage. City of Duncanville, Texas
www.ci.duncanville.tx.us/purchasing_bid_tabs.shtm

Bruce, Tiffany. Email Interview. City of Flower Mound, Texas. 22 January 2007

City of Allen, Texas. Received Bid Tabs. 6 June 2006

City of Arlington, Texas. Received Bid Tabs. 21 June 2006

City of Cedar Hill, Texas. Received Bid Tabs. 19 June 2006

City of Coppell, Texas. Received Bid Tabs. 8 June 2006

City of Desoto, Texas. Received Bid Tabs. 27 June 2006

City of Euless, Texas. Received Bid Tabs. 16 June 2006

City of Flower Mound, Texas. Received Bid Tabs. 19 June 2006

City of Fort Worth, Texas. Received Bid Tabs. 24 January 2007

City of Plano, Texas. Received Bid Tabs. 14 June 2006

City of Richardson, Texas. Received Bid Tabs. 7 June 2006

City of Saginaw, Texas. Received Bid Tabs. 19 June 2006

Doty, Jimmy. Email Interview. Hanson Pipe. 9 February 2007

Engineering Department Bid Tab List webpage. City of Grand Prairie, Texas
www.gptx.org/PlanningAndDevelopment/Engineering/BidTabList.aspx

Flanigan, Chris. Email Interview. City of Allen, Texas. 26 January 2007

Green, Bink. Email Interview. American Cast Iron Pipe. 7 February 2007

Simmons, Spencer. Personal Interview. US Pipe. 2 February 2007

Optimal Scheduling of Pipe Replacement, Including Opportunity, Social and Environmental Costs

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Summary: The renovation period for a pipe is usually calculated by minimizing the sum of the renovation and maintenance costs. In this paper, the optimum period is calculated in a similar way although including additional costs of increasing importance that usually are not taken into account (water lost through leaks, social costs and opportunity costs). Additionally, the influence of the new trenchless technologies is also assessed. Finally, an example allows for the quantification of the influence of the new factors and the interpretation of the opportunity costs.

Keywords: Pipe renovation, cost analysis, trenchless technologies.

1. Introduction

The problem at stake, due to its importance, has been tackled by many researchers. Amongst these, the work by Shamir and Howard (1979) must be highlighted due to its later impact. The authors, admitting an exponential increase in failures with time, obtained the optimum renovation period by calculating the minimum value of the sum of renovation and repair costs. Other authors (Loganathan et al., 2002; H.P. Hong et al., 2006) have followed a similar procedure, even though they admit that the number of failures follows a non-homogeneous Poisson distribution. Kleiner et al. (2001) include other renovation criteria such as the social costs derived from lower standards of service.

Generally speaking, two are the main costs to be taken into account. The renovation costs per meter of main (C_1) and the analogue repair and maintenance costs (C_2). The first ones decrease with time, for the renovation cost is considered constant and the longer the pipe lasts, the smaller the yearly cost in net present value is. The second ones are calculated assuming a time evolution of the failure index and an associated unit cost for repairs.

These analyses ignore some factors which may become significant with time, for instance the cost of the water lost through leaks. In order to correctly assess the influence of these emerging factors in the optimum renovation period, additional costs have to be taken into account. More specifically, the variable costs of water (C_3), social costs (C_4) and the opportunity costs (C_5). Each one of these terms can be broken down in several parts.

For instance, C_3 is the sum of the variable costs of the water lost through leakage (production and environmental costs, C_{31}), and the additional energy costs required to pressurize leaked water as well as the resulting from an increased roughness, with passing time, in the pipe (C_{32}).

The social cost, C_4, includes two terms. The first one C_{41} -often ignored- relates to the impact created by the repair works (such as traffic interruptions) while the second one C_{42} considers the penalties derived from missing a level of service target (for instance maintaining the standard operating pressure).

Finally, the opportunity cost[1] C_5, is associated to the savings derived from renewing the pipe while performing other utility or road works which are more urgent. As a consequence costs of a certain importance are shared (e.g. machinery, staff, tools, etc.). The savings can even reach the total cost of the installation if other works are in charge of digging and replacing the pavement.

It is quite obvious that the result of the analysis will depend on whether these additional costs are included or not, particularly when the variable cost of water, C_{31}, is high (e.g. desalinated water).

2. Fundamentals

As previously stated, the two main costs considered by Shamir and Howard (1979), C_1 and C_2, have different behavior patterns. These same principles apply to the new costs proposed in this paper. For instance, the social and opportunity costs depend on the installation technology. Table 1 summarizes the notation used and characterizes both the type of cost and the influence of the installation technique on its value.

The pipe renovation cost can be divided into C_{11} and C_{12}. The price of the pipe depends on the pipe material, while the installation costs are related to the

[1] In this paper, opportunity cost makes reference to the savings obtained when at least part of the installation works are carried out by another company. This concept goes beyond the traditional one designating the cost of investment of the available resources taking advantage of a certain economic opportunity versus other available options (in other words versus the value of a better option not chosen)

installation techniques. The repairs and maintenance costs, C_2, are sensitive to the number of failures and consequently to pipe aging. The cost of water loss through leakage C_{31} and the increase in energy consumption C_{32} are only dependant on the number of bursts, which also increase with time. The social costs C_{41} basically take into account the disruptions caused by the installation, which are dependant on the technology used. On the other hand, C_{42}, which takes into account the cost of providing a lower standard of service, does not depend on it.

Table 1. Characterization of the costs

Cost	***Year of estimation of the cost***	***Sub-cost***	***Does technology have an influence?***	***Cost nature***
C_1 Renovation	t_p	C_{11} Pipe cost	No	Investment
	t_p	C_{12} Pipe installation	Yes	Investment
C_2 Repairs and maintenance	t_p	C_2	No	Maintenance
C_3 Variable costs related to water	t_p	C_{31} Leakage	No	Maintenance
	t_p	C_{32} Energy losses	No	Maintenance
C_4 Social	t_p	C_{41} Disruptions caused by the works	Yes	Occasional
	t_s	C_{42} Costs related to lower standards of service	No	Maintenance
C_5 Opportunity	t_c	C_5	Yes	Savings in the investment

3. Cost analysis

Determining the optimum renovation period requires the quantification of the time evolution of all costs. In the following analysis, all costs quoted are yearly costs and calculated per meter of mains. This implies that all pipes considered for this analysis need to be homogeneous in age, diameter, material and installation technique used.

Figure 1 shows the time scale for the cost analysis according to Shamir and Howard (1979): t_0 is the first year for which there are available pipe failure data, t_p is the current year, t_r is the renovation year. In addition, t_c represents the year when the installation costs may be reduced due, for instance, to other utility works, and finally t_s is the year in which the service provided falls below the standard level. Once t_r has been calculated, it may happen that either t_c or t_s, or both, may happen later in time. In such case, the initial hypothesis should be re-stated and all values calculated again.

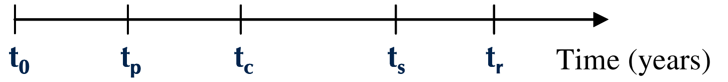


Figure 1. Time scale

3.1. Renovation costs (C_1)

The renovation costs according to Shamir and Howard (1979) are:

$$C_1(t_r) = \frac{C_1}{(1+R)^{tr-tp}}$$

Where C_1 is the pipe renovation cost (/m) and R the discount rate. If the two components of $C_1(t)$ are considered:

$$C_1(t_r) = C_{11}(t_r) + C_{12}(t_r) = \frac{C_{11}}{(1+R)^{tr-tp}} + \frac{C_{12}}{(1+R)^{tr-tp}}$$

3.2. Maintenance and repair costs (C_2)

The total costs of maintaining and repairing the pipe from the current year until the replacement year is:

$$C_2(t_r) = \sum_{t=tp}^{tr} \frac{C_m(t)}{(1+R)^{t-tp}} = \sum_{t=tp}^{tr} \frac{C_b \cdot N(t_0) \cdot \exp(A \cdot (t-t_0)))}{(1+R)^{t-tp}}$$

Where C_b is the unit cost of repairing a burst. Additionally, t is a generic year between t_p and t_r, and $N(t_0)$ is the number of bursts per length of main in the reference year t_0. Finally A is the annual growth rate of the number of bursts.

3.3. Variable costs related to water (C_3)

The yearly volume of water lost through leaks is assessed by considering an average unit leakage flow rate q_f, and an average time of duration for the leak, Δt_a. Considering these factors, the volume lost through leaks is:

$$V_f(t) = q_f \cdot N(t_0) \cdot \exp(A \cdot (t-t_0)) \cdot \Delta t_a$$

And consequently the total cost of the leakage volume (C_{31}) from the current year until the replacement year is:

$$C_{31}(t_r) = \sum_{t=tp}^{tr} \left(\frac{q_f \cdot N(t_0) \cdot \exp(A \cdot (t-t_0))}{(1+R)^{t-tp}} \right) \cdot \Delta t_a \cdot C_W$$

Where $C_{31}(t_r)$ is the total accumulated cost associated to the leakage loss volume (until the renovation is undertaken in the year t_r). Then, q_f is the average volume lost per leak (from the current year to the renovation year) and Δt_a is then considered to be half the inspection period for the pipe (during the pipe' life half the interval between sweeps is commonly used). Finally, C_w are the total water related costs in (€/m^3), resulting from the production and the environmental costs.

The cost associated to the energy consumption (C_{32}), is:

$$C_{32}(t_r) = k \cdot \left[\sum_{t=tp}^{tr} \frac{\left(\gamma \cdot \left(q_f(t) \cdot N(t_0) \cdot \exp(A \cdot (t - t_0) \cdot \Delta t_a \right) \cdot \frac{p_s}{\gamma} \right) \cdot C_E \cdot \frac{1}{\eta}}{(1+R)^{t-tp}} \right]$$

Where p_s is the operating average pressure and C_E the cost of the consumed energy in €/Kwh. The efficiency of the pumps is η and k a coefficient defined by Colombo and Karney (2003) quantifying the increase in pressure needed to compensate the existence of leaks (k>1).

3.4. Social costs derived from disruptions caused by the works (C_{41})

C_s represents the social costs derived from the disruption created by the repair works (which is similar to a one time investment, for instance C_1). The net present value is:

$$C_{41}(t_r) = \frac{C_S}{(1+R)^{tr-tp}}$$

This term contains the costs related to traffic disruptions, damage to the pavement and other infrastructures, loss of productivity, business losses, community complaints, increased costs of cleaning services, etc.

4. Other occasional costs

This point covers costs which may, or may not, appear in the latter analysis depending on when and which the solution replacement is finally adopted. When applicable, these costs should be included as indicated here.

4.1. Social costs related to service levels below the standards of service

The social costs due to lower levels of service (for instance low pressure) are to be faced every year between t_s and t_r. A first estimate of these costs would imply a constant penalty, resulting in:

$$C_{42}(t_r) = \sum_{t=ts}^{tr} \frac{C_P}{(1+R)^{t-tp}}$$

Where C_p is the yearly penalty due to the fact that the standards of service are being missed.

4.2. Opportunity costs (C_5)
The opportunity costs can be rendered as a benefit or a negative cost. The opportunity cost may appear in a certain moment, year t_c, and consequently needs to be treated as a step function.

Its maximum range of variation is $0 < C_5 < -C_{12}$, since the term relative to the cost of installing the pipe C_{11} will always be the same.

Traditional techniques of installation would take the value closer to the upper limit, while new trenchless technologies would bring it down to the lower limit.

5. Determination of the optimum renovation period and the minimum cost

The sum of all costs (except C_{42} and C_5 which are not always applicable) is:

$$C_T \equiv C_{11}(t_r) + C_{12}(t_r) + C_2(t_r) + C_{31}(t_r) + C_{32}(t_r) + C_{41}(t_r) \qquad (1)$$

The optimum renovation period t_r is calculated by minimizing the total cost function $\frac{dC_T}{dt_r} = 0$, which substituted in (1) will allow the determination of the minimum cost of the works, $C_T(t_r^*)_{MIN}$. Grouping as (1) the three investment costs C_{11}, C_{12} and C_{41}, with an analogue behavior in time (in € per length unit):

$$C_{11}(t_r) + C_{12}(t_r) + C_{41}(t_r) = \frac{C_{11} + C_{12} + C_S}{(1+R)^{tr-tp}} = \frac{I}{(1+R)^{tr-tp}}$$

It is also convenient to group the annual cumulative costs (C_2, C_{31} and C_{32}):

$$C_2(t_r) + C_{31}(t_r) + C_{32}(t_r) =$$

$$\sum_{t=tp}^{tr} \frac{C_b \cdot N(t_0) \cdot \exp(A \cdot (t - t_0)))}{(1+R)^{t-tp}} + \sum_{t=tp}^{tr} \left(\frac{q_f \cdot N(t_0) \cdot \exp(A \cdot (t - t_0))}{(1+R)^{t-tp}} \right) \cdot \Delta t_a \cdot C_W +$$
$$+ k \cdot \left[\sum_{t=tp}^{tr} \frac{\left(\gamma \cdot \left(q_f(t) \cdot N(t_0) \cdot \exp(A \cdot (t - t_0) \cdot \Delta t_a \right) \cdot \frac{p_s}{\gamma} \right) \cdot C_E \cdot \frac{1}{\eta}}{(1+R)^{t-tp}} \right]$$

Finally obtaining:

$$C_2(t_r) + C_{31}(t_r) + C_{32}(t_r) = M \cdot \sum_{t=tp}^{tr} \frac{\cdot N(t_0) \cdot \exp(A \cdot (t - t_0)))}{(1+R)^{t-tp}}$$

Where:

$$M = C_b + \left(q_f \cdot \Delta t_a \cdot \left(C_W + \frac{k \cdot p_s}{\eta} \cdot C_E \right) \right)$$

With M being an annual "maintenance" cost (also in € per length unit) resulting from pipe aging. In fact includes repair plus water and energy losses.

Taking all these factors into account, the function (1) to be minimized is:

$$Min(C_T(t_r)) \equiv Min\left(\frac{I}{(1+R)^{tr-tp}} + M \sum_{t=tp}^{tr} \frac{N(t_0) \cdot \exp(A \cdot (t - t_0)}{(1+R)^{t-tp}} \right)$$

And the optimum renovation period:

$$t_r^* = t_p + \frac{1}{A} \ln \left(\frac{I \cdot (\ln(1+R)}{M \cdot N(t_0)} \right) \qquad (2)$$

A result which quite resembles the one obtained by Shamir and Howard (1979) but with a wider meaning in the new maintenance and investment parameters M and I. Additionally, the installation technique used influences t_r^* through C_{12} and C_{41}.

6. Example

The numerical example that follows allows you to quantify the influence of the new costs in the problem at stake. The starting data for a polyethylene pipe are:

Cost C_1

Diameter 300 mm, length 1 m, discount rate R = 2%. Costs associated to the renovation (conventional trench and insertion, breaking the pipe with a hydraulic system) as described in Table 2.

Table 2. Price of the polyethylene pipe (€ in the current year)

Diameter (mm)	***C_{11} (€/m)***	***C_{12} (€/m)***	***Total cost (€/m)***
300 (with trench)	70.56	258.57	329.13
300 (trenchless)	70.56	238.44	309.00

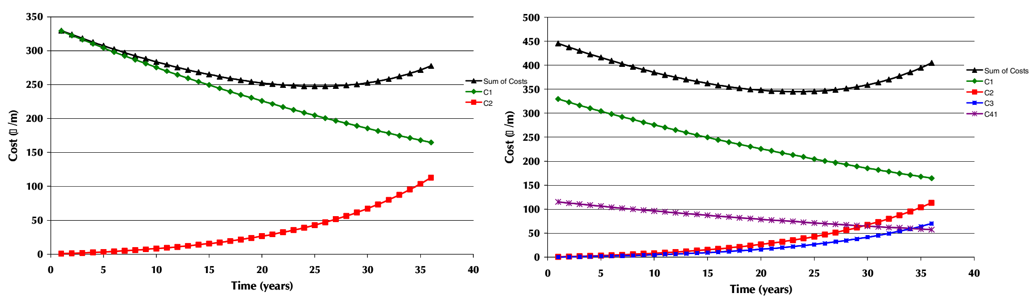
Cost C_2
Cost of repairing a single leak = 1400 €
Nt_0 = 40 (failures/year/100Km). In the present paper, the current year is the same for which there are available failure data, $t_0 = t_p$.
An annual rate of growth of the number of bursts (1/year). A = 0.1.

Cost C_3 (variable cost related to water loss C_3)
$C_w = 0.3$ (€/m^3) Total cost (production and environmental).
$q_f = 15$ (m^3/day) Average volume lost per leak and day.
$\Delta t_a = 182.5$ days. (the network is swept yearly)
$\frac{p_s}{\gamma}$ = 30 m.w.c (meters of water column). Average pressure in pipes with leaks.
$C_E = 0.1$ (€/Kwh) Pumping energy costs.
$\gamma = 9810$ N/m^3 Specific weight of water.
$\eta = 0.8$ Efficiency of the equipment.
K = 1.4 Energy adjustment coefficient due to leaks.

Cost C_4
$C_{41} = 115$ €/m Social costs, with trench
$C_{41} = 27$ €/m Social costs, no trench
$C_{42} = 2$ €/m Social costs (penalty due to levels below service standards, with $t_s = 10$)

Cost C_5
$C_5 = 15$ (€) Amplitude of the step function of the opportunity cost.

6.1. Influence of the new costs
It was logical to expect that presence of the new costs leads to a shorter renovation period. Figure 2a shows the traditional costs, while Figure 2b shows all considered costs. Finally Figure 3 shows the results. It can be seen that the new costs shift the first curve upwards (higher costs) and towards the y axis (the renovation period is shortened). The optimum renovation period is reduced from 24 to 22 years.



Figures 2 a y 2 b. Time variation (traditional and total costs)

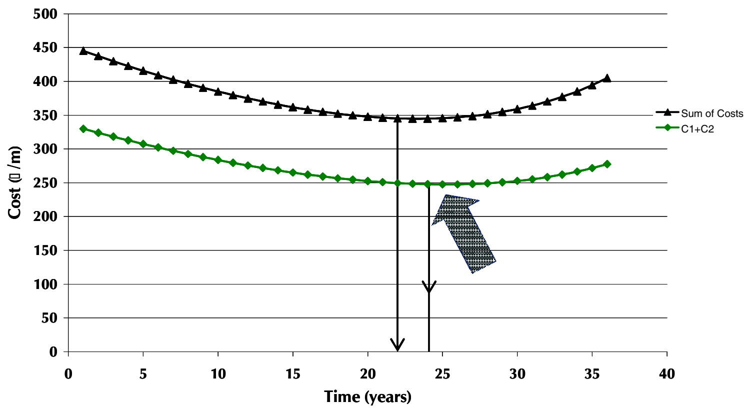


Figure 3. Optimum renovation period (traditional costs vs. Total costs)

6.2. Influence of the installation technology

The installation technology influences the curves, and consequently also affects the optimum renovation period. Figure 4 shows the results for all costs when changing the installation technology. Using a cheaper option reduces the optimum period. As a matter of fact, it is further reduced from 22 to 20 years.

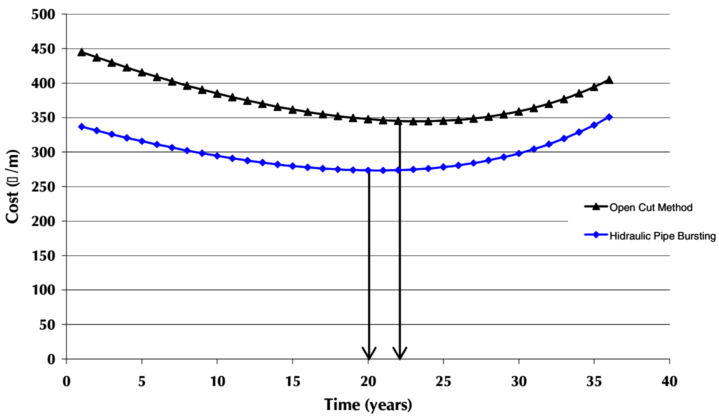


Figure 4. Influence of the installation technology (all costs included)

6.3. Treatment of occasional costs

The social costs generated by missing targets of standards of service or by opportunity costs may or may not become additional costs. When considered, the first one is integrated in the cost structure from year t_s (with $t_s < t_r$) while the second one (negative opportunity cost) only plays a role if the works are carried out in the year t_c. In the first case, missing the standards of service implies that the social costs increase, as reflected by Figure 5. If the pipe is replaced with trench, and taking into account all costs, it is supposed that the standards of services are not met from year

10. The applied penalty further reduces the renovation period, leaving it in t_r = 16 years.

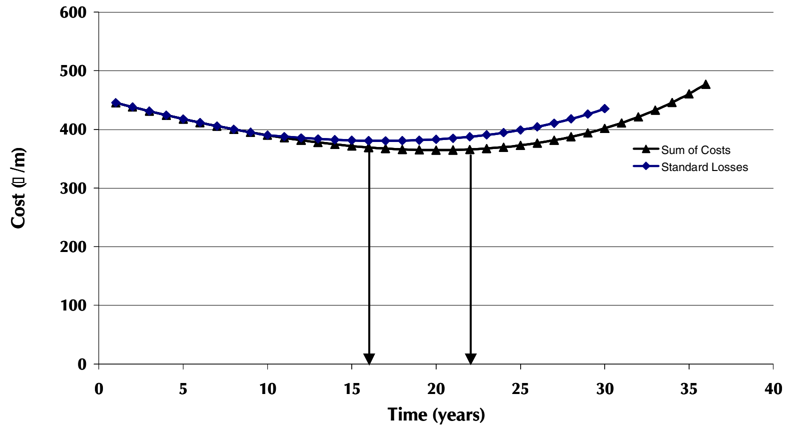


Figure 5. Variation of the total costs when the standards of service are not met

If the opportunity arises of carrying out the works in conjunction with other utilities during the year t_c (always with $t_c < t_r$) the costs curve (Figure 6) is shifted downwards a distance equal to the negative opportunity cost C_5. The comparison between the displaced curves (for the different values of C_5) and the original curve shows the number of years which would be reasonable to anticipate the works. And hence, for savings of 5 €/m, the works should be anticipated from t_r = 16. Higher values (C_5 = 10 ó 15) advice a greater anticipation (t_r = 14 or 13).

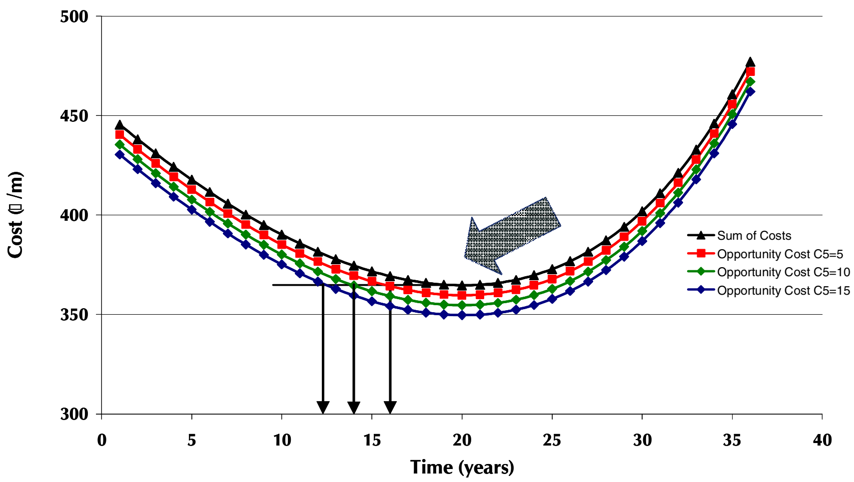


Figure 6. Time shift resulting from opportunity.

7. Conclusions

Qualitatively, the previous analysis bears little surprise. When the new costs rise significantly with time (for instance, water loss through leaks) the optimum renovation period is anticipated, the more with the higher costs. From another perspective, the costs associated to the renovation of the pipe increase the investment, and consequently extend the optimum renovation period. Regarding the technology used, the cheaper the sooner the renovation should be performed. Finally the opportunity costs may anticipate the renovation of the pipe a certain number of years.

Quantitatively, it seems quite obvious that the expressions to assess the new costs must be adapted to each context. The ones included in this paper allow considering with a certain degree of accuracy the influence of these costs which are usually neglected. In any case, and according to previously established objectives, it seems clear that the water loss through leakage must be taken into account, for the production and environmental costs are significant, and they reduce considerably the optimum renovation period.

8. References

Colombo, A. F. and Karney, B. W. (2002). "Energy Cost of Leaky Pipes: Toward Comprehensive Picture". *Journal of Water Resources Planning and Management*, 128(6); 441-450, November 1, 2002.

Hong, H. P., Allouche, E.N. and Trivedi, M. (2006). "Optimal Scheduling of Replacement and Rehabilitation of Water Distribution Systems." Journal of Infrastructure Systems, 12(3): 184-191. September 2006.

Kleiner, Y, Adams, B. J. and Rogers, J.S. (2001). "Water Distribution network renewal planning." Journal of Computing in Civil Engineering, Vol. 15, No 1, January, 2001.

Logathan, G.V. , Park, S. and H. D. Sherali (2002). "Threshold Brake Rate for Pipelines Replacement in Water Distributions Systems". Journal of Water Resources Planning and Management, 128(4): 271-279. July/August 2002.

Shamir, U., and Howard, C. D. D. (1979). "Analytical approach to scheduling pipe replacement." Journal of AWWA, 71(5), 248-258

Study on Mechanical Property of Corroded Pipeline

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ABSTRACT: Stochastic finite element method (FEM) was applied to analyze the mechanical property of the corroded pipeline. Firstly, mechanical properties of the corroded steel were studied. Then, dynamic properties of the corroded pipeline were investigated; sensitivity analyses were performed on the effective factors respectively. Considering the distribution characters of the defects on the pipe surface as stochastic variables, a series of numerical investigations were executed on the corroded pipeline. Relation between the dynamic property of the submarine pipeline and the corrosion state of the material is achieved.

KEY WORDS: Corroded pipeline; mechanical property; statistical distribution function; stochastic FEM; factor analysis

1. Introduction

The construction of submarine pipelines for delivering gas and oil is increasing with the exploration and utilization on the offshore oil/gas resource in China. However, since salt content of seawater is about 3 percent, and the seawater is a kind of strong electrolyte, pipelines servicing in the marine environment will be corroded by the seawater and the marine atmosphere. Furthermore, Sulfur dioxide and carbon dioxide will take corrosive effects on the structures as well. Meanwhile, many environmental loads acting on the pipelines, e.g. wave, flowing of the fluid etc., will accelerate the corrosion effects on the deterioration of the structure. Although some methods (such as anticorrosion coating below the concrete coating and catholic protection using sacrificial anodes) have been accepted to protect the modern pipeline from corrosion, submarine pipelines in service always showed early deterioration. By now, the failure of the submarine pipeline has led to the broken-down of the oilfield production, and has caused the pollution of the surrounding sea area. Therefore, as a large submarine transport network, safety of the pipeline has become to be a critical issue and attracted more and more attention by scientists and engineers worldwide, and the evaluation of the integrity of the corroded pipeline has long been and continuous to be a concern.

Since 1970s, improved condition assessment methods for the pipeline systems have been continuously seek for. The original work was sponsored by AGA-NG18 in early 1970, and a semi-empirical equation for remaining strength of corroded pipelines was developed by Battelle (Bjornoy and Marley,2001). Modifying the formula proposed by Battelle, some other methods such as Shell criterion, RSTRENG, etc have been suggested. However, both Battlle's formula and the various modified methods are just taking the single corrosion metal mass loss defects in pipelines exposed to internal pressure into account, regardless of the interaction of defects and additional loads, etc. Based on the risk and reliability principles, Nes(2001) established a methodology for pipeline condition assessment.

To develop a rational evaluation of the present state of the submarine pipeline, it is essential to make a reliable study on the mechanical property of the deteriorating structures. The mechanical property of the corroded submarine pipelines has been investigated (Fu 1995,Batte 1997,Moke 1991,Zhu 2003). In a summary, two aspects of deficiencies exist in their studies: 1) Since the corrosion is a stochastic process, both the mechanical property of the material and the structure were always affected by many stochastic factors, such as, the corrosive medium around the pipeline, corrosion defects on the metal (such as depth, number and the distribution law of the defects, et al), and the additional loads, etc. However, the stochastic character has not been taken into account in the present studies. 2) For the corroded in-service submarine pipeline, elastic stage will be shortened and the structure will enter into plastic stage earlier during the loading process, which will result in the early occurrence of surface cracking, and the cracking of the concrete will accelerate the corrosion of the metal. Since the effective factors are complex, and coupling effect always exists among the factors, it is difficult to take all the factors into account simultaneously. The present studies are limited to single metal loss defect, and excludes interaction of the defects and other loads.

In this paper, stochastic finite element method (SFEM) will be applied to analyze the mechanical property of the corroded pipeline. Firstly, the effects of the defects on the steel were studied. Taking characters of the defects on the metal as stochastic variables, a series of numerical investigations of corroded submarine pipelines were performed. The changing tendency of the dynamic property of the submarine pipeline with the corrosion state of the material is achieved.

2. Mechanical property of the rusty steel

2.1. Steel corrosion character in concrete

Corrosion mechanism of the steel in concrete is an electrochemistry process, which had been studied widely. To describe the corrosion state of the steel in concrete, a kind of NDT instrument (GXY-1A) was applied for the corroded reinforced concrete beams (depth of the concrete cover is 20mm), which had been corroded severely by Chloride for more than 10 years. A potential map of RC beam is shown in Fig.1, in which the figures indicate the potential value of the measuring point. If the potential value bellows –400, it is indicated that the reinforcement in the area have been severely corroded.

Fig.1 NDT instrument (GXY-1A)

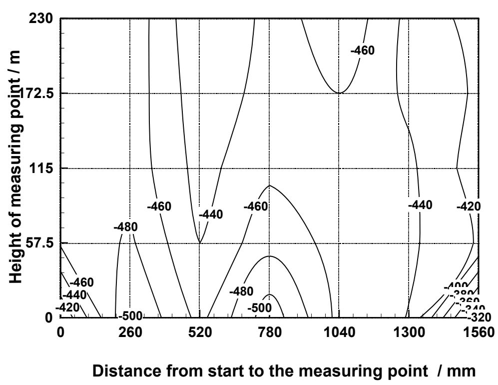


Fig.2 Potential maps of RC beam corroded by Chloride

From Fig.2, it is shown that chloride has a strong corrosive effect on the RC structure, and lead to occurrence of the corrosion pit on the steel surface.

2.2. Mechanical property of the defected steel

The commercial finite element software, ANSYS, was applied to study the mechanical property of the steel. 2D finite element model of the steel with a diameter of 18mm is shown in Fig.3. Values of 340MPa, 520MPa, 200Gpa and 0.3 were used for the yield strength, the ultimate strength, Modulus of Elasticity and Poisson’s ratio respectively in the analyses. Idealistic elastic-plastic constitutive model of the steel is applied in the analysis, which is shown in Fig.4.

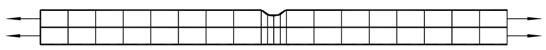
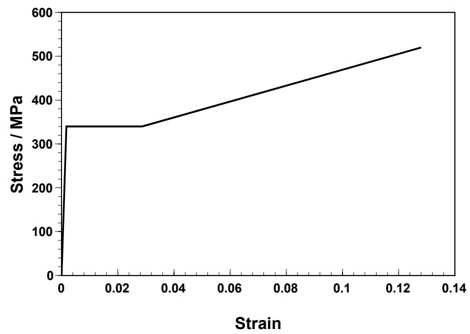
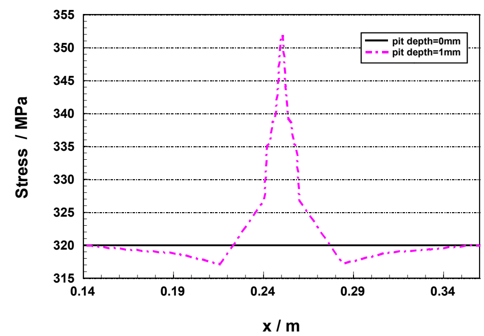


Fig.3 FE model of corroded steel

Assuming that there exists a corrosion pit with the depth of 1mm on the steel surface, the stress distribution along the longitudinal direction under the yield load of the intact steel can be obtained in Fig.5.

From Fig.5, it is shown that the existence of corrosion pit will result in significant stress concentration around the pit, and will cause the degradation of the strength of the damaged steel as well.





Note: x is the distance from the corrosion pit to the end of steel

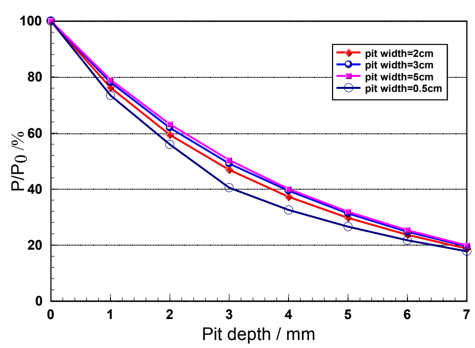
Fig.4 Constitutive model of the steel Fig.5 Stress distribution along the tensile direction

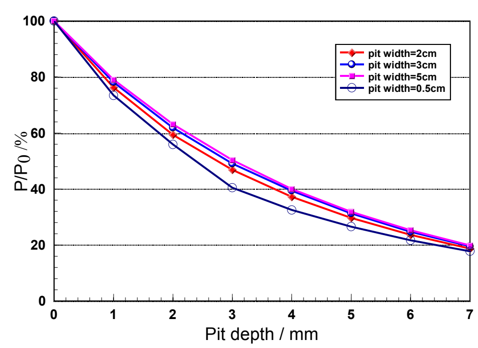
2.3. Sensitivity analysis of the effective factors

According to the corrosion mechanism of the steel in concrete, the corrosion pit always takes on 3 types of shape: shallow, intermediate and deep corrosion defects. The mechanical property of the damaged steel is always affected by many factors, such as the shape, location and maximum depth of the corrosion pit, distribution and the number of the maximum corrosion pit, the steel diameter, etc. Sensitivities of these parameters on the mechanical property of the damaged steel are investigated respectively as follows.

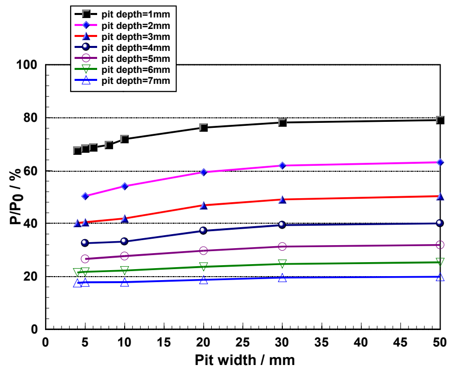
2.3.1. Effect of the pit shape on the mechanical property of the defected steel

Considering the corrosion pit on the steel surface with the width change from 4 to 50mm and the depth change from 1 to 7mm, the yield load of the defected steel can be numerated respectively. Therefore, the relationship between the pit shape (width and depth) and the normalized ultimate load P/P_0 (P and P_0 are the yield load of the defected and the intact steel respectively) can be achieved (Fig.6).





(a) Effect of the depth of corrosion pit on the yield load of rebar (b) Effect of the depth of corrosion pit on the yield load of rebar



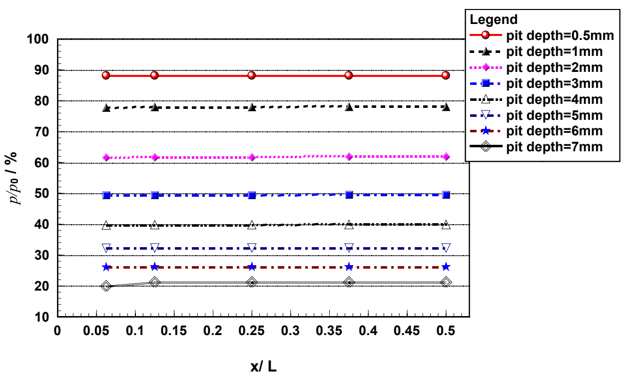
(c) Effect of the width of corrosion pit on the yield load of rebar

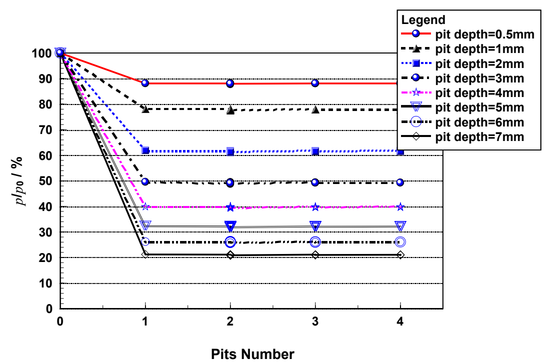
Fig.6 Effect of corrosion pit on the yield load of defected steel

From Fig.6, it can be concluded that the degradation of the defected steel is mainly determined by the depth of corrosion pit, while the width of the corrosion pit have a relative little effect on the mechanical property of the damaged steel.

2.3.2. Effect of the maximum pit location on the mechanical property of the defected steel

Since that the maximum pit depth is the key effective factor, effect of the location of the maximum pit on the defected steel is discussed herein. Considering the depth of the pit range from 0.5 to 7mm, the location of the pit move from the end of the steel to the middle of the steel, the yield load of the steel can be numerated respectively. The relationship between the pit locations versus the normalized ultimate load can be obtained. (Fig.7)





Note: x is the distance from the corrosion pit to the end of steel; L is the length of the steel

Fig.7 Effect of the pit location on the yield load of steel Fig.8 Effect of the number of pit on the yield load of steel

The result shows that the pit location has a minor effect on the yield strength of the defected steel.

2.3.3. Effect of the number of the pit on the mechanical property of the defected steel

To study the effect of the number of corrosion pits on its mechanical property, the defected steel with 1 to 4 pits (the pit depth is 0.5, 1, 2, 3, 4, 5, 6 and 7mm) were numerated respectively. Relationship between the pit number and the normalized ultimate load is plotted in Fig.8.

From the numerating results, it is evident that that the pit number have little effect on the load strength of the defected steel.

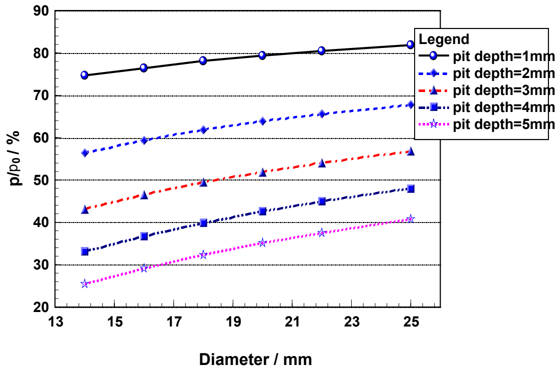
2.3.4. Effect of the steel diameter on the mechanical property of the defected steel

Steel diameter is another effective factor for the mechanical property of the defected steel as well. Steels with 6 diameters (14, 16, 18, 20, 22, 25mm), and the pit depth is 1, 2, 3, 4, 5mm for each kind of steel were analyzed respectively. Relationship between the steel diameter and the normalized ultimate load can be obtained under each pit depth (Fig.9 (a)). Furthermore, the relationship between the ratio of pit depth to diameter and the normalized ultimate load is given in Fig.9 (b).

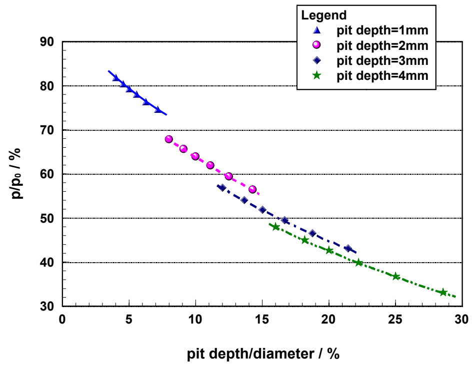
Therefore, relation between the yield load and the relative depth of the rusty rebar (pit depth/rebar diameter) can be derived,

$$p = \left[a_1 - a_2 x + a_3 x^2\right] \cdot p_0 \tag{1}$$

where p is the yield load for the rusty rebar, p_0 is the yield load for the sound rebar, a_1、a_2 and a_3 are the effective factors which is relative to the maximum pit depth.



(a) relation between steel diameter and the P/P_0



(b) relation between steel diameter and P/P_0

Fig.9 Effect of the steel diameter on the yield load of steel

Table 1 Calculated value of the factors for different corrosion pit depths

Pit depth / mm	a_1	a_2	a_3
1	94.0417	3.454	0.10368
2	85.9302	2.514	0.03144
3	81.212	2.357	0.02709
4	75.25578	1.99703	0.01828

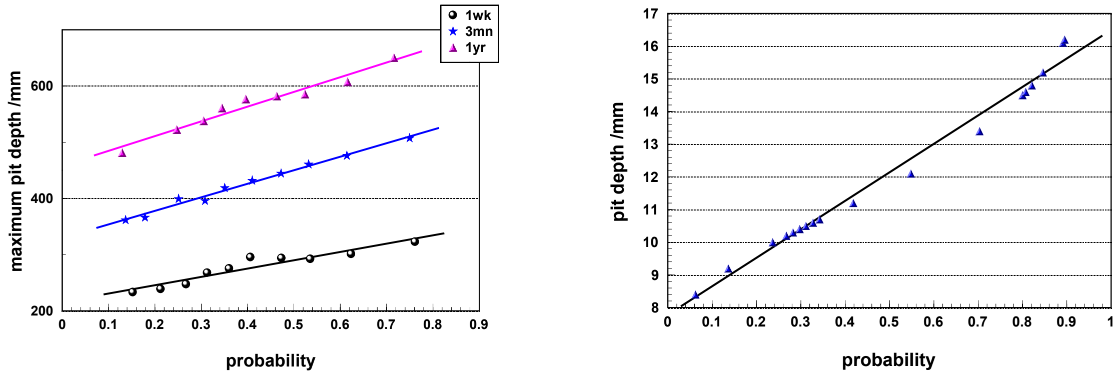
From the above, it can also be derived that with the deepen of the corrosion pit, relation between the yield load and the relative depth will be close to be linear.

2.4. Stochastic analysis of the defected steel

Using the experimental data of Aziz, which Al alloy corroded in the water for 1 week, 3 months, 1 year respectively, Finleg derived the probabilistic distribution rule of the maximum corrosion pit depth (Fig. 10(*a*)). From the corrosion experiment of alloy of Al-Mg in 3.5％ solution of NaCl, Xiao (1994) applied statistical method analyze the distribution rule of the maximum pit depth on the alloy surface (Fig. 10(*b*)). It is discovered that the distribution of the maximum depths of pits accords with the Gumbel first approximating function. The extreme value law can be aptly used to evaluate pit corrosion problems. Probabilistic distribution function can be expressed as,

$$F(x) = \exp\{-\exp[-(x-\mu)/\sigma]\} \tag{2}$$

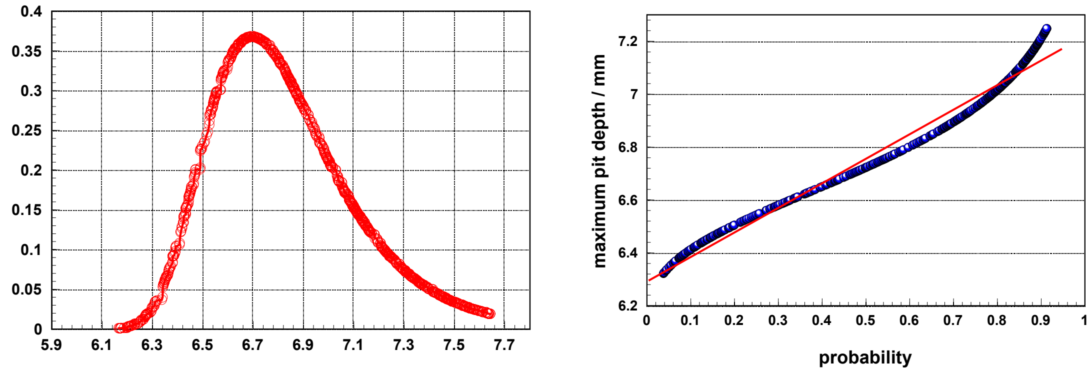
where μ is the location factor, σ is the scalar factor, x is the maximum pit depth.



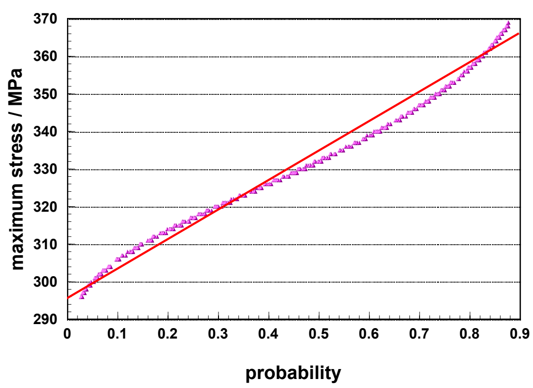
(a) corrosion of Al Alloy in water (b) corrosion of Al-Mg Alloy in NaCl

Fig.10 Probabilistic distribution function of the maximum corrosion pit depth on metal surface

However, lacking of experimental data led to the high difficulty to determine the location factor and scalar factor exactly by statistical analysis up to now. In this paper, the mean value μ and the standard deviation σ are assumed to be 6.7mm and 0.8mm respectively. A sample set with a sample size of 750 was generated by Matlab, the probabilistic density distribution of the sample is shown in Fig.11 (a). Statistical distribution of the maximum stress along the steel can be achieved (Fig. 11 (c)).



(a) probabilistic density distribution of the maximum pit depth (b) probabilistic distribution of the maximum pit depth



(b) probabilistic distribution of the maximum stress

Fig.11 Probabilistic distribution of the maximum corrosion pit depth and maximum stress

From the numerating results, it is indicated that if the maximum depth of pits obeys to extreme value law, the distribution of the maximum stress of the rebar will accord with the Gumbel first approximating function.

The probabilistic distribution function $F(x)$ can be written as,

$$F(x) = \exp\{-\exp[-(x-324.13)/21.93]\} \tag{3}$$

Where, $F(x)$ is the probabilistic distribution function, and x is the maximum stress of the rebar.

3. Mechanical property of the corroded pipeline

3.1. Corrosion description of the submarine pipeline

According to the design discipline of the submarine pipeline, the steel pipe is covered by concrete. The steel pipe is the main part for load bearing, while the concrete is the secondary part. Cross section of the pipeline is plotted in Fig.12. Interactive action of electrochemistry and the complex environmental loads are the mean causes for the corrosion of the pipelines.

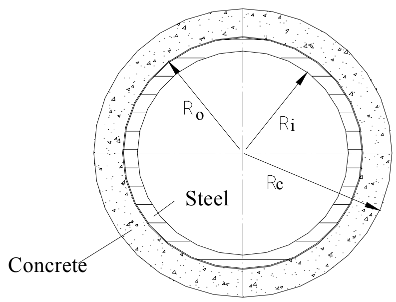
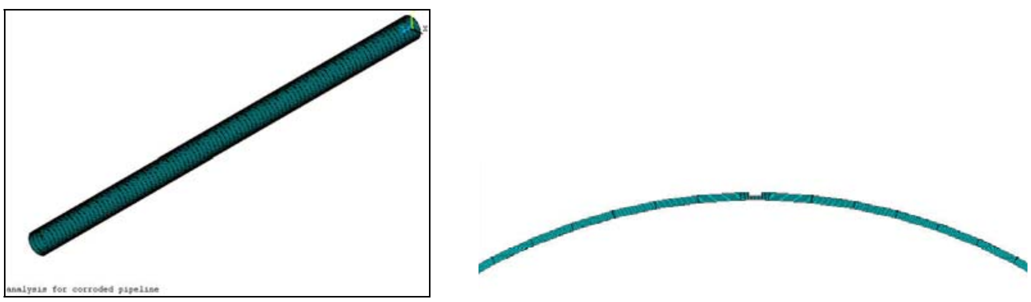


Fig.12 Cross section of the pipeline

3.2. Numerical model

Considering that steel is the major part for the load bearing, and corrosion of the steel is the key effective factor for the mechanical property of the pipeline, the concrete is not taken into consideration herein. In this study, geometry of the pipeline is with an outside diameter of 508mm (20 in.) and average wall thickness of 6.35mm (0.25 in.). Ideal elastic-plastic constitutive model of the steel, which is shown in Fig.3, is applied in this analysis as well. The commercial finite element software, ANSYS, was applied to study the dynamic mechanical property of the pipeline. The 3D FE numerical model of the pipeline is shown in Fig.13.



(a) FE model of the pipeline (b) defect on the surface of the steel

Fig.13 Numerical model of pipeline

According to the corrosion mechanism of the steel in concrete, the corrosion of the steel pipe is developed from a corrosion pit. Assuming that there exists a corrosion pit on the steel surface, the dynamic property of the pipeline will be studied. Different defect shape are considered, ratio of the defect depth to the pipe wall thickness varies from 0.1to 0.5, ratio of the defect width to the defect width varies from 2 to 10.

3.3. Sensitivity analysis of the effective factors

3.3.1 Effect of the pit shape on the load carrying capacity of the defected pipeline

Considering the corrosion pit on the pipe surface with the depth change from 1/4 to 1/2 of the pipe thickness and the width change from 4 to 8mm, the yield load of the defected steel can be numerated respectively. Relationship between the pit shape (width and depth) and the normalized ultimate load P/P_0 (P and P_0 are the yield load of the defected and the intact steel respectively) can be achieved. It can also be drawn that it can be drawn that the maximum pit depth is the key effective factor to the mechanical degradation of the damaged pipeline.

3.3.2 Effect of the defect shape on the dynamic property of the defected pipeline

Since dynamic loads (such as wave, fluid, earthquake, etc) are the major loads acted on the pipeline, the dynamic mechanical property of the pipeline is investigated in this paper. Based on the finite element model shown in Fig.13, taking the defect width and depth as stochastic variables, the dynamic responses for different free spanning length (20m, 60m, 90m, 120m) in simply supported end condition are calculated. Assuming that the mean value μ and the standard deviation σ of the pit depth are 0.02mm and 2mm, while the values of μ for the pit width are 2~10 times to the value of pit depth, a sample set with a sample size of 1000 was generated with direct Monte Carlo Simulation method. Probabilistic distribution of the corrosion pit depth is shown in Fig.14. Length of the defect along the longitudinal direction ranges from 5 time of the defect width to the length of the pipe. Modal analysis was performed respectively, and the frequencies of the structure can be extracted. Statistical distribution of the structural frequencies of the pipeline can be achieved.

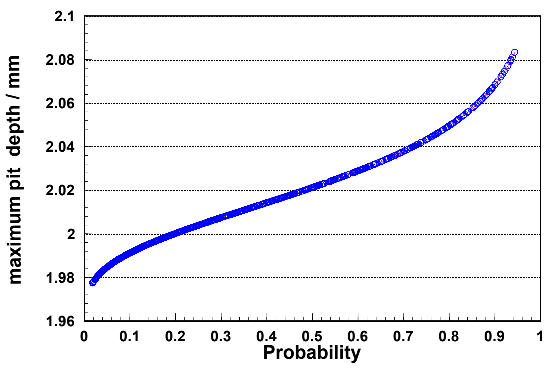


Fig.14 Probabilistic distribution of the corrosion pit depth

The results for each length of the defect show a similar trend, that is, both the single defect shape and the length of the pipe have minor effect on the natural frequency of the damaged structure. Even if the defect runs through the whole pipe, just a slight change occurs on the structural frequency, which is less than 0.5%.

3.3.3 Effect of the defect location on the dynamic property of the defected pipeline

Considering the depth of the pit range from 1/10~1/2t, the location of the pit rotate from the top of the pipe to the bottom of the pipe, and the pit move from the end to the middle of the pipe. Modal analysis was executed on the defected model respectively, and the structural frequencies were achieved. All the results show the same tendency, that the frequency has little change with the different defect location.

From the above study, it can be drawn that neither the location nor the quantity of the damage has distinct effect on the structural frequency.

4. Conclustions

Taking a kind of corrosion measuring instrument (GXY-1A), corrosion state of the steel in concrete is depicted. Sensitivities of the factors (such as the width, the depth, the location, the number of the corrosion pit, etc) on the mechanical property of the damaged steel are analyzed respectively. Simulating the corrosion as a pit on the steel surface, numerical investigation is carried out on the corroded pipeline in this paper. From the results, the following conclusions were drawn:

(1) The corrosion pit is the key factor that results in the degradation of the damaged steel;
(2) From the sensitivity analysis of the effective factors for the mechanical properties, it can be drawn that the maximum pit depth is the key effective factor to the mechanical degradation of the damaged steel, while the width, location, and number of the pit all have a minor effect on the mechanical property of the damaged steel;
(3) Taking the maximum depth of the corrosion pits as random input variable, and assuming that the distribution of the pit depth obeys to normal distribution, the maximum stress is numerated by SFEM. From the numerating results, it is indicated that the maximum stress will obey to normal distribution if the pit depth obeys to normal distribution;
(4) Taking both the maximum depth and the width of the corrosion pit as random input variable, it is concluded that the single defect shape has minor effects on the natural frequency of the damaged pipeline.

From the numerating results, it is discovered that the pipe length, the defect location and the defect number all take minor effects on the natural frequency of the damaged pipeline. As a result, it will be difficult to fulfill the health condition assessment of the in-service pipeline by the structural frequency obtained from the dynamic monitoring.

Acknowledgements

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References

Hroar Nes and Lars Sortland (2001). "Risk-based condition assessment and inspection planning for submarine pipeline system." *Int Offshore and Polar Eng Conference*, ISOPE, Stavanger, Norway

Bjornoy OH and Marley MJ (2001). "Assessment of corroded pipelines: Past, Present and Future." *Int Offshore and Polar Eng Conference*, ISOPE, Stavanger, Norway

Bjornoy OH, Sigurdsson, G, Marley MJ (2001). "Background and development of DNV-RP-F101 “corroded pipelines”." *Int Offshore and Polar Eng Conference*, ISOPE, Stavanger, Norway

Fu B, Kirkwood MG (1995). "Predicting failure pressure of internally corroded linepipe using the finite element method." *OMAE*, VolⅤ,pp 175-184

Batte AD, Fu B, Kirkwood MG and Vu D (1997). "New methods for determining the remaining strength of corroded pipelines." *OMAE*, VolⅤ, pp 221-228

Moke DHB, Pick RJ, Glover AG and Hoff R (1991). "Bursting of line pipe with long external corrosion." *Int. J. Pres. Ves. & Piping*, Vol 41, pp 195-216

Zhu XK and Leis BN (2003). "Strength criteria and analytic predictions of failure pressure in corroded line pipes." *Int Offshore and Polar Eng Conference*, ISOPE, USA

Zivica V. (2000). "Utilization of electrical resistance method for the evaluation of the state of steel reinforcement in concrete and the state of its corrosion." *Construction and Building Materials*, Vol 14, pp 351-358.

Li Q. (1987). Application of probability statistical method to evaluate localized corrosion. *Corrosion and Protection*, No 1, pp 32-36.

Xiao Heng, Li Jiuqing. (1994). "The statistico-analysis for the bit testing data of the Al-Mg alloy." *Journal of University of Science and Technology Beijing*, Vol 16, No 3, pp 97-102.

Wastewater Collection System Rehabilitation and Replacement (R&R) Program Prioritization

By John T. Caldwell, P.E., CDM

Introduction

A large Florida Utility identified the need to develop and implement a wastewater collection system rehabilitation program. CDM provided services to provide technical and program management assistance to staff in developing and implementing a countywide Wastewater Collection System Rehabilitation and Replacement (R&R) Program. This project developed a systematic methodology to identify, rank and prioritize sewer system repairs and/or replacement projects. The process developed an initial project list and the development of a wastewater collection system rehabilitation program implementation strategies and tools. The program provided an evaluation of the gravity system, pump stations, and forcemains in order to prioritize R&R needs.

In order to identify the immediate rehabilitation needs in the system and to provide a framework for a continuing future rehabilitation strategy, collection system components (gravity systems, pump stations, and forcemains) were evaluated based on both criticality (consequence of failure) and condition (probability of failure). Prioritizing pipes and pump stations in terms of their criticality and condition identified components that needed immediate inspection, rehabilitation, and continued monitoring.

Prioritization Framework

One way of identifying pipes and pump stations that will receive the most immediate inspection or rehabilitation is to rank pipes in terms of their criticality (or consequence of failure) and condition (probability of failure). Pipes and pump stations whose failure creates a large impact on the community and environment and whose condition is the poorest will receive immediate inspection and rehabilitation. Pipes and pump stations that receive a lower criticality and condition rating will receive some level of continued monitoring but no immediate action or rehabilitation.

Criticality Factors

Pipe or pump station failure has an impact on transportation, business, the environment, the public, and repair crews no matter where it occurs. The purpose of the ranking system is to differentiate the pipes and pump stations in terms of their consequence of failure. All pipes and pump stations in the system will receive some level of monitoring, rehabilitation, or immediate action. The goal is to match up the pipes and pump stations an appropriate level of maintenance and rehabilitation. For example, pipes identified as being very critical in terms of their transportation, urban, environmental, or public impact would receive a high frequency of inspection, maintenance or rehabilitation. Actual Factors included:

Pump Stations	
Diameter/Size	Downstream Forcemain Diameter in inches – surrogate of flow per pump station
Urban Impact	Proximity to major or minor road.
Environmental Impact	Proximity to water body larger than 4 acres.
Population Density	Use 2005 projected wastewater flow divided by subarea acreage.
Emergency Power Capability	Does it have a generator (Y/N)?
Gravity Sewers	
Diameter/Size	Average diameter = Sum(Lineal Ft. X Diameter)/Total Lineal Ft.
Urban Impact	Proximity to major or minor road. (same result as pump station)
Environmental Impact	Proximity to water body larger than 4 acres.
Population Density	Use 2005 projected wastewater flow divided by subarea acreage.
Emergency Repair – Depth of Gravity Line	Calculated depth from rim elevation to first invert elevation. Used average depth for subarea.
Groundwater Table	Depth to water table. West (1), Southwest (2), South (3), Southeast (4), and East (5)
Force Mains	
Diameter/Size	Diameter in inches
Urban Impact	Proximity to major or minor road.
Environmental Impact	Proximity to water body larger than 4 acres.
Population Density	Use 2005 projected wastewater flow divided by subarea acreage.

Condition Factors

In addition to criticality factors, each pipe or pump station will be ranked based on condition. Those portions of the system that are in poor condition have a higher probability of failure and, therefore, should be higher priority for investigation and repairs. Condition will be assessed based on three factors: structural condition, maintenance frequency, and capacity. Actual Factors included:

***Pump* Stations**	
Age	Using asset date from sewer structure geodatabase table data. Property appraisal – Earliest date structure was developed by subdivision.
Number of SSOs	SSOs by pump station per subarea
Number of Trouble Calls	Trouble calls per subarea.
Capacity	Capacity at 2020 Projected Peak Flow.
Gravity Sewer	
Material	Gravity Pipe Material – Predominant pipe material of subarea.
Age	Property appraisal – Earliest date structure was developed by subdivision.
Number of SSOs	SSOs per subarea
Number of Trouble Calls	Trouble calls per subarea

Preventative Maintenance	Frequency of preventative maintenance. None, (1) Annually (3), Bi-annually (4), and Monthly (5)
Capacity - I/I Concerns	Unknown Data – all data assigned 3.1
Force mains	
Material	Predominant forcemain material
Age	Property appraisal – Earliest date structure was developed by subdivision.
Number of ARVs	Number of ARVs per 100 feet
Number of Line Breaks	Number of line breaks in last 5 years and number of line breaks per 1,000 feet
Number of Trouble Calls	Trouble calls per subarea

Factor Levels

Each of the factors had a range of levels assigned for each pumps station, gravity main collection area, or forcemain segment. The purpose of assigning levels is so that pipes and pump stations can be differentiated in terms of their condition or criticality. The following table shows an example of the levels assigned for just one gravity main collection area factor used to score the Size criticality factor:

Average Gravity Main Diameter per Collection Area	Levels Assigned
</= 8"	1
> 8" to </= 12"	2
> 12" to </= 18"	3
> 18" to </= 24"	4
> 24"	5
Average Diameter for Gravity collection area equals the sum of length x diameter divided by total length	

Overriding Factors were also used that override all other factors when as specific issue or condition exists. For example, whenever a gravity sewer main collection area was found to have more than 50% of the pipe material to be vitrified clay, it would override all other condition scores. Other overriding factors included forcemains located in Backyard/Side-yard Utility Easements, Pump stations and Gravity Mains with known gravity back-ups into homes, and forcemains in close proximity or under major highways around the area.

Project Prioritization

Data Collection and Analysis

Data was collected for use in the prioritization framework using a variety of sources including:

- Latest Wastewater master plan information for flow projections

- GIS data for pipeline attributes and proximity to water bodies and proximity to major roadways
- Maintenance management system for trouble call and line break work orders
- Listing of historical SSO's noted in a collection system area

From the data collected, each pump station, gravity main collection area and forcemain element was evaluated and received a score for each factor. Each factor was assigned a level from 1 to 5. Each factor score was then used to calculate the condition and criticality rating for each pipe and pump station. The calculations were done by using the level assigned to each factor and the relative importance of each factor. Each factor was given a percentage of importance as shown on the following criticality scoring example for gravity main collection areas:

	15%	20%	20%	10%	15%	10%	10%	**100%**
Gravity Main Collection Area	Size	Trans /Urban Impact	Env. Impact	Popul. Density	Depth of Gravity Line	Ground water Table	Subarea Flow	**Weighted Criticality Score**
30	1	3	5	3	2	1	2	**2.5**
33	2	5	3	1	3	4	1	**2.9**
48	3	1	1	2	4	1	2	**1.8**
49	4	3	5	4	5	3	3	**3.7**

The combination of condition and criticality ratings was used to determine priorities for repair or replacement of system assets. For each pump station, gravity main collection area, and forcemain element, the final weighted criticality and condition scores were added together. The highest scoring pump stations or pipes would receive higher priority for renewal or rehabilitation.

Cost Estimating

Cost estimates were developed for each of the systems in this study. These costs were, at best, planning level in nature, in that many assumptions relating to system rehabilitation and/or replacement were made.

The cost functions use for Forcemains and Gravity Systems were expressed as a cost per linear foot of piping, for each piping diameter. These pricing factors were composite figures, including costs of materials, installation, and peripherals such as man-holes for gravity systems. A 35% contingency was also added in for engineering costs.

The cost estimates for Pump Stations were tabulated based on pump type, i.e. duplex, triplex, or quadraplex.

Project Lists

Project priority lists were generated for each set of systems. Each list details the scoring, cost estimates, and any comments / notes about each system element. The scoring was broken down to each of the contributing factors for reference.

The Pump Station and Gravity System lists also include adjusted scores per operations input. The Condition score was adjusted for the Pump Stations based on either current project status (decreased score for recent or current rehabilitation / repair) or problem stations identified by operations staff (increased condition per comments).

The Forcemain list included groupings of potential projects. These sets of pipe segments were based on similar pipe material / size within a proximate area or pipe run. Total costs for a group are also listed, as well as a running total for yearly budgeting / planning.

Structural Condition Models for Sewer Pipeline

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Abstract

Proper management of sewer pipeline networks involves fulfillment of many technical requirements under economic constraints. Therefore, cost effective solutions are required to assist municipal engineers in prioritizing maintenance and rehabilitation needs. This demands a systematic approach to condition assessment of rapidly deteriorating sewers. Performance evaluation of sewers through random inspections is expensive. Therefore, there is an urgent need to develop a proactive sewer pipeline condition prediction methodology. This paper presents a method for assessing a sewer's structural condition by utilizing general pipeline inventory data. Based on historic condition assessment data, condition prediction models for sewers are developed using multiple regression technique. The final outcome of these models produces most likely condition rating of pipes, which will assist municipal agencies in prioritizing pipe inspection and rehabilitation to critical sewers.

Introduction

The condition of underground sewer infrastructure across North America has been deteriorating day by day. The maintenance and rehabilitation of aging sewers have become an overburden in terms of budget allocation and investment planning for municipalities. Multiple objectives may exist for planning budget allocation that could be dependent upon certain constraints and available resources. This makes the task of planning, prioritizing and allocating funds a complex exercise (Ruwanpura et al. 2004).

Investment in sewer rehabilitation must be based on inspection and evaluation of sewer condition. However, random inspection of sewers is extremely expensive. Therefore, due to budget constraints, only 22% of Canadian municipalities have a complete condition assessment program (Rahman et al. 2004). Thus, it is important to prioritize inspections to those sewers which are more vulnerable to deterioration phenomena and have higher risk of collapse for proactive sewer rehabilitation planning.

There are two main avenues of improvement in sewer rehabilitation planning (Ariaratnam et al 2001):

1) Collection and storage of adequate inspection information regarding current condition of sewer system
2) Ability to predict sewer deficiency prior to failure in order to facilitate timely sewer inspection and repair

Therefore, predicting structural condition of sewers should be the first consideration of municipal managers in order to prioritize detailed inspections. In this context, current paper describes the development of an improved methodology for analyzing and interpreting historical sewer data. The methodology involves the use of this data in developing multiple regression models to predict the existing condition rating of sewers. The application of these multiple regression models would provide decision makers with a means to prioritize inspections of sewers, which have higher risk of failure.

Previous Decision Support Tools for Sewer Condition Assessment

There are numerous documented studies that focus on various aspects of drainage systems including different methodological approaches to predict the condition of drainage pipes (Ruwanpura et al. 2004). In terms of objectives, these studies can be categorized into two classes. The first approach is to help municipal engineers in preventing inadequate sewer inspections through developing automated condition assessment techniques. The second strategy is to develop proactive tools for prioritizing inspections to critical sewers.

There are many examples of studies for developing automated systems for the interpretation of sewer inspection data. Moselhi et al (2000) described image analysis and pattern recognition techniques of sewer inspection, based on neural network analysis of digitized video images. The neural network analysis technique was found helpful in identifying four categories of sewer defects: cracks, joint displacements, reduction of cross-sectional area, and spalling. Chae at al (2001) developed an automated sewer inspection data interpretation system. Artificial neural networks were used to recognize various types of defects in sewers through optical data obtained from inspection with Sewer Scanner and Evaluation Technology (SSET). Sinha et al (2006) presented an algorithm for the automated analysis of scanned underground pipe images. The algorithm consisted of image pre-processing followed by a sequence of morphological operations for the classification of different sewer defects: cracks, holes, joints, laterals, and collapsed surfaces.

The other approach is to predict a sewer's existing condition prior to its detailed inspection for selective, cost effective sewer inspection. Hasegawa et al. (1999) developed a method for condition prediction of sewers on the knowledge of pipe material, length, diameter and other characteristics. However, it was concluded that the method could not evaluate sewer's condition effectively. Ariaratnam et al (2001) developed logistic regression model for condition evaluation of sewers. The model was developed through historical data based upon factors; such as, pipe age, diameter, material, waste type and depth. Another approach for condition assessment of large sewers was developed by assessing the impact of different factors; such as location, size, burial depth, functionality etc., on sewers (McDonald et al 2001). Similarly, Baur et al (2002) developed a methodology of forecasting condition of sewers by using transition curves. These transition curves were developed through the historical condition assessment data. Sewers characteristics; such as, material, period of construction, location were used to define the existing condition of sewers for scheduling detailed inspection. Yan et al (2003) proposed a fuzzy set theory based approach for a pipe's condition assessment. Various linguistic factors: soil condition, surroundings, etc, were transformed through fuzzy theory into numerical format for assessing their impacts on pipes. Ruwanpura et al (2004) used rule based simulation methodology to predict condition rating of sewers. The model predicted the condition rating of pipe based on age, material and length of pipe. Najafi et al (2005) developed an artificial neural network model for predicting the condition of sewers based on historical data.

The above mentioned approaches tend to predict existing condition of sewers for prioritizing detailed inspections. This paper suggests a simple and easy to use approach towards condition prediction of sewers. The paper describes the development of multiple regression models in this regard. These models will provide adequate knowledge about condition of sewers in order for municipal agencies to optimize cost of sewer inspection.

Sewer Pipe Deterioration

Although pipelines are designed for a particular lifespan under standard operating condition, their deterioration never follows a set pattern (Najafi et al 2005). Pipe deterioration is a very complex process and related to various pipe characteristics such as pipe material, period of construction, location, diameter and gradient (Yan et al 2003). In general, these pipe characteristics or factors can be divided into three categories; physical, operational and environmental. Table 1 shows the subdivision of these factors into further categories. It also explains how these factors contribute in pipeline deterioration phenomena.

Physical factors comprise general pipe characteristics such as length, diameter etc. While operational factors deal with flow performance in which they adapt operational and maintenance strategies. The third category is related to certain environmental factors directly influencing a pipe's criticality and deterioration. These factors include type of surrounding soil, traffic volume above pipe, etc.

Data Collection and Variables Selection

The effectiveness of condition rating regression model depends upon the quality of collected data and selection of predictors. Data are collected from two municipalities; Pierrefonds (Quebec, Canada) and Niagara Falls (Ontario, Canada). The collected data included general pipeline inventory record, AutoCAD drawings and CCTV inspection reports. One of the major problems in the preliminary analysis of data is that both municipalities have adapted different sewer pipeline condition grading systems. Data from Niagara Falls consist of WRc (Water Research Centre UK) classification system, while the other data set is based on CERIU (Centre for Expertise and Research on Infrastructures in Urban Areas, Canada) classification system. As WRc classification system is known as the "Embryo Codes" for world wide sewer rehabilitation industry (Thornhill et. al 2005), all data from Pierrefonds are converted into WRc classification system for generalizing the model building approach.

Table 1: Factors that Contribute to Sewer Pipeline Deterioration

Factors		Explanation
Physical	*Pipe Length*	Pipe in longer length and having greater length to diameter ratio are more likely to suffer from bending stresses
	Pipe Diameter	Small diameter pipes are more susceptible to beam failure
	Pipe Material	Pipes manufactured with different materials show different failure patrons.
	Age	More probability of collapse for aged pipes
	Average Depth	If the depth is very low, the pipe is susceptible to surface live load. If depth is high, the pipe is susceptible to overburden. Moderate depths increase the life of sewers
	Pipe Gradient	Steeper slopes of pipe cause high flow velocity which increases erosion phenomena
Operational	*Infiltration/ Exfiltration*	Infiltration and exfiltration wash soil particles which reduces the soil support along the pipe
	M & R Strategies	Good maintenance and repair strategies increase the service life of sewers
Environmental	*Type of waste*	Different types of waste react with different pipe materials in a different manner causing pipe erosion
	Type of Soil	Different types of soils provide side supports to pipes according to their own physical and chemical properties
	Bedding Conditions	The chance of pipe failure increases with improper bedding condition of pipes
	Frost Factor	The load on buried sewers increases due to additional frost load in winter
	Other Utilities	Proximity of other underground installations increases the criticality of a sewer
	Traffic Volume	The bending stresses in the pipe increase with the increase in live load above pipe

The next step was data sorting for selection of input variables. These variables which have maximum historical information are selected for further model development.

The predictors chosen are pipe material, pipe material class, diameter, length, age, depth, bedding material class and street categories. The information collected regarding bedding material class and street category is described in a generalize manner in order to facilitate regression model application.

Table 2: Bedding Material Classes as per BRE and OPSD Standards and their transformation weights for model development (Adapted from Perkins, 1974 and Zhao et al 2001)

Bedding Class	Description	Bedding Factor B_f		
		BRE	OPSD	Model Input Weights
A	Reinforced Concrete Cradle or Arch	3.4	2.8	4
	Plain Concrete Cradle or Arch	2.6		
B	Well Compacted Granular Material	1.9	1.9	3
C	Well Compacted Backfill	1.5	1.5	2
D	Flat Sub Grade	1.1	--	1
Others	Cement Stabilized Material	2.6 to 3.4	--	----

Five different types of bedding material have been specified by Building Research Establishment (BRE) UK, which are also acceptable in USA (Perkins, 1975). In Ontario, Canada, OPSD (Ontario Provincial Standard Drawings) defines four classes of bedding material (Zhao et al 2001). These classes have been defined on the bases of bedding factor B_f. In general, the bedding factor B_f is defined as (Zhao et al 2001):

$$B_f = \frac{W}{S_{eb}} \qquad \text{---------------------- Equation 1}$$

Where, W is calculated external load and S_{eb} is 3 – edge bearing strength.

The bedding factors for specific classes in both classification systems have been shown in Table 2 where the input weights are also shown. The weights have been allotted according to material class.

Not only is the bedding factor redefined, but also is the average annual daily traffic (AADT) data. The AADT data depends upon location of streets and other factors. Therefore, instead of using the locally available AADT data (as shown in table 3), the local streets are categorized according to American Society of Civil Engineers (ASCE) Classification. These classifications along with their assigned input weights for model development are shown in Table 3. The AADT data is based on Niagara Fall's classification.

Design and Diagnostics of Structural Condition Models

In linear regression models, a linear relationship is assumed between the response variable (Y) and the several independent variables (X1, X2, X3...). The major point in these models is that the combined effect of all variables on the dependent variable is investigated rather than individual relations between dependent and independent variables (Dikmen et al 2005). However, linear regression model includes not only first – order models in predictor variables but more complex models as well. Consequently, model with transformed variables or with different interaction terms should be considered as linear regression models due to their respective linear parameters (Kutner et al 2005). Therefore, different consideration for different functional forms of predictors in regression model is the first key step in model development.

Table 3: Transformation of Traffic Data into ASCE 1990 Urban Street Classification System and Model Input Weights

ASCE Classification	Description	Approximate AADT (Niagara Falls)	Input Weight for Model
1	Arterial	10,000 - 12,500	4
2	Collector	7,500 - 10,000	3
3	Sub-collector	5,000 - 7,500	2
4	Access	< 5,000	1

Initially, in order to check possible interactions and multicolinearities among variables, matrix plots are developed for the input data. When two predictive variables in a regression model are highly correlated, they both convey essentially the same information. Therefore, their interactions and relationships among themselves have to be examined carefully. In the process, if matrix plot shows some possible interactions, the decision to include or exclude variables from the model is made through best subset analysis. Figure 1 shows an example of the best subset analysis for one trial.

In Figure 1, it is clear that the model with higher R^2, R^2-adjusted, lower S, and closer C_p to number of variables is the most appropriate model. Where, S is the standard deviation of residuals. In this case, the variable "pipe depth" should be excluded from the model (C_p = 6.0 & S = 0.83087). After diagnosing correlation, the developed models are checked for their statistical validity. The main diagnostics in this regard are R^2, P(F), P(t), residual diagnostics, and LOF. The R^2 (co-efficient of multiple determination) measures the proportional variation in structural condition explained by sewer's attributes; age, diameter, material, length etc. The results shown in Table 4 illustrate that 72% to 82% of the total variability in structural condition can be explained through the developed regression equations. The R^2-adjusted accounts for the number of predictors in the model. Both values indicate that the model fits the data well.

To determine P(F) for the whole model, a hypothesis test is carried out. The null hypothesis (H_0) assumes that all regression coefficients, β_0, $\beta_{1\ldots}$ βp_{-1} are zero i.e. $\beta_0 = \beta_1 = \beta p_{-1} = 0$. The alternate hypothesis (H_a) assumes that not all of them equal to zero. Based on the Minitab's output the p-values for the test are 0.000 for all chosen models (Table 4). This means that null hypothesis is rejected. Similarly, to determine the validity of regression coefficient individually, "t-tests" are performed separately for the β_0, $\beta_{1\ldots}$ βp_{-1}. In case of β_0, the null hypothesis (H_0) of t-test assumes that β_0 =

0; while alternative hypothesis (H_a) assumes that $\beta_0 \neq 0$. Similarly, the other null hypothesis assumes that $\beta_1 = 0$ and vise versa. The results of these tests, for all the three models, shown in table 4, indicate that the p-value for intercept is 0.000, 0.041 & 0.003, respectively. As a result, alternative hypothesis is accepted. Note that for performing F and t tests, the confidence interval α is assumed to be 0.05; that means that null hypothesis can be accepted if the p-value is equal to or greater than 0.05.

Vars	R-Sq	R-Sq(adj)	Mallows C-p	S	Depth	Length	Age	Dia	CC	Bedding	Traffic
1	22.1	21.1	65.4	1.1332				X			
1	19.6	18.6	69.8	1.1512							X
2	40.6	39.0	34.4	0.99645				X	X		
2	34.2	32.4	45.8	1.0487					X		X
3	50.8	48.8	18.2	0.91332				X	X	X	
3	49.3	47.2	20.8	0.92672		X		X	X		
4	56.2	53.8	10.4	0.86734		X		X	X	X	
4	55.6	53.2	11.4	0.87287		X			X	X	X
5	60.1	57.3	5.4	0.83326		X		X	X	X	X
5	58.0	55.0	9.3	0.85546		X	X		X	X	X
6	60.9	57.6	6.0	0.83087		X	X	X	X	X	X
6	60.2	56.8	7.3	0.83842	X	X		X	X	X	X
7	60.9	57.0	8.0	0.83676	X	X	X	X	X	X	X

Figure 1: An Example of Best Subset Analysis for a Trial Model

Similar procedure is performed to check the validity of other regression coefficients associated to each predictor in all regression models. The overall results of t- test are found satisfactory. Some of the t-test results shown in the Table 4 have p values greater than α (0.05). This indicates that there could be a weak evidence of null hypothesis for that particular coefficient. However, due to large number of predictors in models and due to satisfactory results of other statistical diagnostics, these results are concluded as acceptable.

The next step is to check the residual diagnostics. Figure 2 shows an example of residual plots for a trial model. The normal probability plot shows that there could be a possibility of outliers in the data. After a logically based reexamination of data, it is concluded that the points are not outliers and these scenarios could exist. Therefore, the possibilities of outliers are rejected.

Figure 2 also shows the fitted value plot for the model under consideration. In ideal scenario, constant data would be distributed evenly across the plot. That would show the consistent variance across the fitted value range. However, figure 2 shows diagonal bands across the centre line. The reasons for these types of results could be due to:

- Important variable(s) might be omitted from the model (Kutner et al 2005)
- Data variability issues: data composed of integer variables (Anderson et al 2005)

The careful examination of data in hand shows that both the abovementioned possibilities exist in this case. Some of the important variables which could have a strong effect on the existing pipe condition could be missing. For example, type of soil, maintenance and repair history, infiltration etc are important parameters, which affects the existing pipe conditions directly. As information regarding these kinds of parameters is not available; studying the effect of these parameters on the pipe condition is recommended for future research. Data variability could be explained in the case under consideration as the model has integer predictors; concrete class, bedding class factor and street categories. Therefore, the discrete values of these predictors could cause the problem of unequal variance. Consequently, the results are concluded as satisfactory.

Table 4: Summary of Statistical Test Results for Selected Models

Model	**R^2 (%)**	**R^2 (Adj.) (%)**	**P (F)**	**P (t)**							**Durbin-Watson Statistics**	**P (F) Lack of Fit**	
				β_0	β_1	β_2	β_3	β_4	β_5	β_6		**Pure Error**	**Data Sub-setting**
Concrete Pipes	72.7	70.5	0.000	0.000	0.064	0.000	0.010	0.000	0.014	0.000	0..95 $D<d_L$	-	0.0 49
Asbestos Cement Pipes	82.4	78.3	0.000	0.041	0.001	0.034	0.093	0.085	--	--	1.43 $d_L \leq D \leq d_U$	0.8 74	$\geq$ 0.1
PVC Pipes	81.8	78.6	0.000	0.003	0.000	0.008	0.021	0.000	0.000	--	1.82 $D > d_U$	--	0.0 56

The developed models are further investigated through statistical measures such as Durbin-Watson test for auto-correlation and lack of fit (LOF). The Durbin-Watson test considers the null hypothesis (H_0) that there would not be any auto correlation among predictors. The alternative hypothesis (H_a) considers that there is a significance of auto correlation among the predictors. The results shown in Table 4 indicate that H_a should be rejected in case of PVC model, and the test is inconclusive in case of asbestos cement model. The results of concrete model indicate that there could be an evidence of H_a. However, this test is more conclusive for maximum of five predictors; so the results shown for concrete pipe model are not accurate, because in this case the predictors are 6.

The results of LOF test indicate that there are not much replications available to perform the routine pure error test for concrete and PVC models. Therefore, an approximate lack of fit data subsetting test, developed by Minitab® statistical software, is performed on these models. The null hypothesis H_0 is that model fits the data, and alternative hypothesis H_a is that model does not fit data. The decision criterion in case of data subsetting test is that p-value should be equal or greater to 0.01 for an ideal fit model i.e. for the significance of null hypothesis. Table 4 shows a significance of Ha; however, this test is an approximation of pure error test and these models were giving satisfactory results for other statistical diagnostics. Therefore, the models are accepted on the basis of their overall performance of all necessary statistical diagnostics. In case of asbestos cement model, the lack of fit test results for pure error and data subsetting tests are found satisfactory.

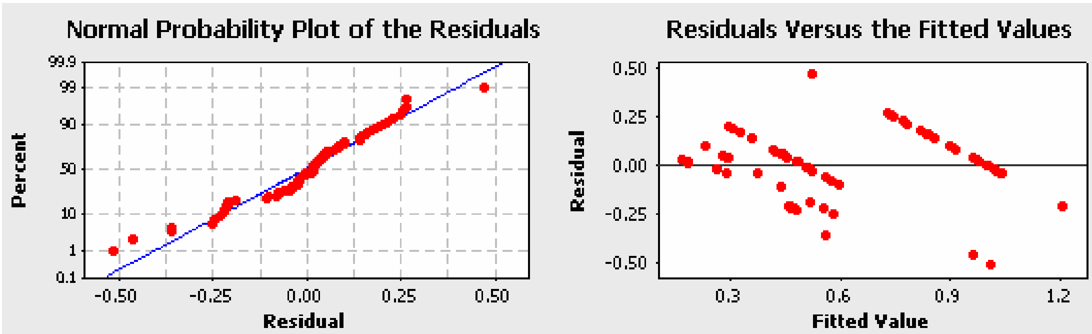

Figure 2: Normal Probability of Residuals and Residual vs. Fitted Value Plots for a Trial Model

After all statistical diagnostics the three models selected for validation are as follows:

Concrete Pipe Structural Condition Prediction Model

$$\left(\frac{1}{Structural\_Condition\_Grade}\right) = 3.94 + 0.592\frac{Log_{10}Diameter}{Length} - 0.00681e^{Street\_Category}$$
$$-3.22Log_{10}Depth - 1.6\frac{Log_{10}Age}{Concrete\_Class} + 6.92\frac{Log_{10}Depth}{Bedding\_Factor} - 5.75\frac{1}{Bedding\_Factor}$$

-------------------- Equation 2

Asbestos Cement Pipe Structural Condition Prediction Model

$$(Structural\_Condition\_Grade)^2 = 20.9 + 542\frac{Log_{10}Depth}{Length} + 0.207Age$$
$$-0.742Asbestos\_Cement\_Class - 14.8Diameter^{0.1}$$

-------------------- Equation 3

PVC Pipe Structural Condition Prediction Model

$$(0.1)^{Structural\_Condition\_Grade} = 2.25 - 0.00642Age - 1.89Length^{0.01}$$
$$-0.0302Bedding\_Factor - 0.0405Street\_Category - 0.000013(Diameter)^{0.3}(Depth)^4$$

-------------------- Equation 4

The values of structural condition grades are according to WRc classification system. The structural condition grading varies from 1 to 5 as per WRc protocols; where 1 is for a pipe in excellent condition and grade 5 means that collapse for the pipe is imminent.

Conclusions:

A methodology for predicting a sewer's structural condition information through the use of historical data is proposed. To assess and predict structural condition of existing buried sewers, multiple regression technique is used. Three different regression models are designed for three different sewer pipe materials: concrete, asbestos cement, and PVC. Various forms of variables are experimented during the design procedure and the best possible scenario is selected for further validation. The selected models are validated through all possible measures to ensure their appropriateness.

It is observed that the selected predictor variables for condition rating model are not enough to completely explain the variation in structural condition of sewers. Therefore, it is recommended that future research should be performed in expanding the model for other pipe attributes, which contributes to sewer's deterioration. The influence of factors such as infiltration, soil condition, and maintenance and repair history, on a sewer's structural condition should be thoroughly investigated. It is further recommended that the model should also be expanded to include other sewer pipe materials; such as clay and bricks sewers etc., to facilitate municipal managers.

It is concluded that the developed technique will assist decision makers in scheduling and prioritizing sewer inspection. Thus, this technique will be helpful in minimizing the cost of random sewer inspection.

References:

American Society of Civil Engineers (ASCE), (1990), "*Residential Streets*", Second Edition, ASCE, National Association of Home Builders of the United states, and the Urban Land Institute (ULI) Publication

Anderson, M & Davenport, N, (2005), "*A Rural Transit Asset Management System*", University Transportation Centre of Alabama Report 04401, USA

Ariaratnam, S, El-Assaly, A & Yang, Y, (2001), "*Assessment of Infrastructure Inspection Needs using Logistic Models*", ASCE Journal of Infrastructure Systems, Volume 7, No 4, December 2001

Barqawi, H, (2006), "*Condition Rating Models for Underground Infrastructure: Sustainable Water Mains*", Master's Thesis, Concordia University, Montreal

Baur, R & Herz, R, (2002), "*Selective Inspection Planning with Ageing Forecast for Sewer Types*", International Water Association (IWA) Journal of Water Science and Technology, Volume 46, No 6-7, page 389-396

Chae, M and Abraham, M, (2001), "*Neuro-Fuzzy Approaches for Sanitary Sewer Pipeline Condition Assessment*", ASCE Journal of Computing in Civil Engineering, Volume 15, Number 1, January, 2001

Dikmen, I, Birgonul, M & Kiziltas, S, (2005), "*Prediction of Organizational Effectiveness in Construction Companies*", ASCE Journal of Construction Engineering and Management, Volume 131, Number 2

Hasegawa, K, Wada, Y & Miura, H, (1999), "*New Assessment System for Premeditated Management and Maintenance of Sewer Pipe Networks*", Proceedings of 8th International Conference on Urban Storm Drainage, Page 586-593, Sydney, Australia

Kutner, M, Nachtsheim, C, Neter, J & Li, W, (2005), "*Applied Linear Statistical Models*", Fifth Edition, McGraw-Hill Inc, USA

McDonald, S & Zhao, J, (2001), "*Condition Assessment and Rehabilitation of Large Sewers*", Proceedings of International Conference on Underground Infrastructure Research, page 361-369, Waterloo, Canada

Moselhi, O and Shehab-Eldeen, T, (2000), "*Classification of Defects in Sewer Pipes using Neural Networks*", ASCE Journal of Infrastructures System, Volume 06, Number 03, September, 2000

Najafi M & Kulandaivel G, (2005), "*Pipeline Condition Prediction Using Neural Network Models*", Proceedings of the American Society of Civil Engineers (ASCE) International Pipeline Conference, USA

Rahman, S & Vanier D, (2004), "*An Evaluation of Condition Assessment Protocols for Sewer Management*", National Research Council of Canada Research Report Number B-5123.6

Ruwanpura J, Ariaratnam, S, and El-Assaly, A, (2004), "*Prediction Models for Sewer Infrastructure Utilizing Rule-Based Simulation*", Journal of Civil Engineering and Environmental Systems, volume 21, No 3, Page 169-185

Thornhill, R & Wildbore, P, (2005), "*Sewer Defect Codes: Origin and Destination*", U-Tech Underground Construction Paper, April, 2005

Perkins P, (1974), "*Concrete Pipes and Pipelines for sewerage and Water Supply*", Cement and Concrete association, London, UK

Sinha, S and Fieguth, P, (2006), "*Segmentation of Buried Concrete Pipe Images", Journal of Automation in Construction*, Volume 15, Pages 47-57

Yan J & Vairavamoorthy, K, (2003), "*Fuzzy Approach for Pipe Condition Assessment*", Proceedings of the American Society of Civil Engineers (ASCE) International Pipeline Conference, USA

Zhao, J. Q & Daigle, L, (2001), "*SIDD Pipe bedding and Ontario Provincial Standards*", Proceedings of International Conference on Underground Infrastructure Research, Kitchener, Ontario, pp 143 – 152

Trenchless Water Pipe Condition Assessment Using Artificial Neural Network

Zong Woo Geem[1], Chung-Li Tseng[2], Juhwan Kim[3], and Cheolho Bae[4]

Abstract

In order to assess the water pipe condition without excavating, artificial neural network (ANN) model was developed and applied to real-world case in South Korea. For the input in this ANN model, 11 factors such as (1) pipe material, (2) diameter, (3) pressure head, (4) inner coating, (5) outer coating, (6) electric recharge, (7) bedding condition, (8) age, (9) trench depth, (10) soil condition, and (11) number of road lanes were used; and, for the output, overall pipe condition index was derived based on 5 factors such as (1) outer corrosion, (2) crack, (3) pin hole, (4) inner corrosion, and (5) H-W C value. For the ANN computing, each factor was normalized into the range of 0 to 1. The ANN model could find better results than those of multiple regression model in terms of statistical correlation between observed and computed data.

Introduction

The quality of drinking water in supply pipelines is one of the biggest issues because water contamination can be a great risk to public health. Although some amount of budget is allocated for the pipe maintenance such as cleaning, rehabilitating, or replacing, more precise assessment for the pipe condition and corresponding operation scheduling are essential in order to maximize the benefit under the limited budget (Loganathan et al, 2002; Dandy and Engelhardt, 2001). In this sense, artificial neural network (ANN) technique can be an efficient and cost-saving methodology to assess the pipe condition before any real action (cleaning, rehabilitating, or replacing) is taken because it can assess the degree of deterioration without real trenching, but utilizing only historical pipe condition data (ASCE, 2000; Oh, 2000).

The ANN technique may be regarded as a regression technique which considers higher nonlinearity among input and output data. So far, the ANN has been used to analyze data in water distribution network modeling, which includes pipe condition assessment using probabilistic neural network (Lee et al., 1998), short-term water demand forecasting (Jain, 2002), pipe condition assessment using back-propagation

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ANN (Geem, 2003), THM formation prediction (Rodriguez et al., 2003), and chlorine decay prediction in the network (Gibbs et al, 2006).

The back-propagation ANN model developed by Geem (2003) considered seven input factors such as (1) pipe material (cast iron pipe (pit cast), cast iron pipe (centrifugal cast), and ductile iron pipe (centrifugal cast)), (2) bedding condition (skilled bedding, medium bedding, and poor bedding), (3) corrosion (small corrosion, medium corrosion, and severe corrosion), (4) temperature (number of days below 0 °C, such as 0, 1, 2, 3, 4, and 5+), (5) trench width, (6) pipe diameter, and (7) age. The model also adopted window-based platform in order to enhance user-friendliness. However, Geem's study has a drawback with respect to data reality. The data used in the study was not real but arbitrarily generated. Thus, the objective of this study is to verify the robustness of the back-propagation model proposed by Geem (2003) by using measured data from real-world.

Back-Propagation ANN Model

The multi layer perceptron (MLP) model using back-propagation algorithm is the most popular ANN model for prediction (Gibbs et al, 2006). The MLP model normally has the structure of three layers (input layer, hidden layer, and output layer) as shown in Figure 1. However, the number of hidden layers can be more than one, and input layer and hidden layers can have bias values. The output values in hidden layer are calculated using Equation 1.

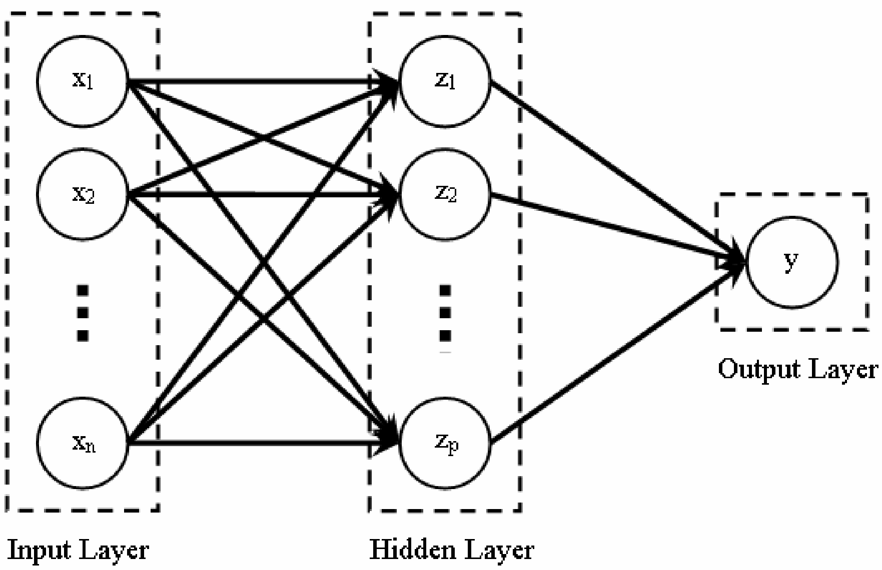

Figure 1. Multi Layer Perceptron Structure

$$z_j = \left(1 + EXP(-1 \times \sum_{i=1}^{n} x_i w_{ij})\right)^{-1}, \quad j = 1, \ldots, p \tag{1}$$

Similarly, the output values in output layer are calculated using Equation 2.

$$y = \left(1 + EXP(-1 \times \sum_{j=1}^{p} z_j w_{j1})\right)^{-1} \tag{2}$$

Output error between observed and computed data is as follows:

$$E = 0.5(d - y)^2 \tag{3}$$

Error signals from output layer and hidden layer are respectively as follows:

$$\delta_y = (d - y)y(1 - y) \tag{4}$$

$$\delta_z = z_j(1 - z_j)\delta_y w_{j1}, \quad j = 1, \ldots, p \tag{5}$$

Adjusted weights between hidden & output layers and input & hidden layers are respectively as follows:

$$\Delta w_{j1} = \alpha \delta_y z_j, \quad j = 1, \ldots, p \tag{6}$$

$$\Delta w_{ij} = \alpha \delta_z x_i, \quad i = 1, \ldots, n; j = 1, \ldots, p \tag{7}$$

Alternatively, momentum is also considered as follows:

$$\Delta w_{j1}^N = \alpha \delta_y z_j + \beta \Delta w_{j1}^{N-1}, \quad j = 1, \ldots, p \tag{8}$$

$$\Delta w_{ij}^N = \alpha \delta_z x_i + \beta \Delta w_{ij}^{N-1}, \quad i = 1, \ldots, n; j = 1, \ldots, p \tag{9}$$

This momentum technique quickens the training (weight adjustment) procedure in flat regions of the error surface and prevent fluctuations in the weights.

Case Study

The proposed MLP model was applied to the trenchless pipe condition prediction using real-world field data. For the input layer, 11 factors such as (1) pipe material, (2) diameter, (3) pressure head, (4) inner coating, (5) outer coating, (6) electric recharge, (7) bedding condition, (8) age, (9) trench depth, (10) soil condition, and (11) number of road lanes were selected out of total 50 factors from the database of 61 records. Out of 11 factors, the following 5 factors have binary values:

1. Inner Coating: 0 = not coated; 1 = coated
2. Outer Coating: 0 = not coated; 1 = coated
3. Electric Recharge: 0 = not recharged; 1 = charged
4. Bedding Condition: 0 = non foundation work; 1 = foundation work
5. Soil Condition: 0 = clay-type; 1 = sand-type

Other 5 factors have normalized continuous values between 0 to 1:

1. Diameter: it is divided by 1,500 (range = 250mm ~ 1,500mm)
2. Pressure Head: it is divided by 12 kg/cm^2 (range = 0.5 kg/cm^2 ~ 12 kg/cm^2)
3. Age: it is divided by 28 years (range = 5 years ~ 28 years)
4. Trench Depth: it is divided by 5.5 m (range = 1 m ~ 5.5 m)
5. Number of Road Lanes: it is divided by 5.5 m (range = 1 m ~ 5.5 m)

For the pipe material, there exist three types: cast iron pipe (CIP), ductile cast iron pipe (DCIP), and steel pipe (SP). In order to consider this value in ANN model, three variables are used as follows:

1. CIP: 1 = pipe is CIP, 0 = otherwise
2. DCIP: 1 = pipe is DCIP, 0 = otherwise
3. SP: 1 = pipe is SP, 0 = otherwise

Thus, total 13 input variables and one bias constant (= 1) are used in input layer.

For the output layer, one variable which represents overall pipe condition is used. The overall condition index was derived based on the following five factors:

1. Outer Corrosion: : 1 = corrupted; 0 = otherwise
2. Crack: 1 = cracked; 0 = otherwise
3. Pin Hole: 1 = punched; 0 = otherwise
4. Inner Corrosion: 1 = corrupted; 0 = otherwise
5. H-W C value: it is divided by 150 (range = 55 ~ 140)

Each of five factors has 20% weighting for the overall pipe condition index which has a value between 0 to 1.

Out of 61 records in the original database, 21 records that do not have any null field for 11 input factors were chosen. For running the ANN model developed in this study, the following parameter values were used:

1. Learning Rate: 1.0
2. Momentum Constant: 0.8
3. Maximum Epochs (Iterations): 100

Figure 2 shows the comparison between real condition score and ANN condition score. The ANN model appears to well represent the data because the statistical value (R-square) between the two is high (0.9433). If one outlier (real score = 0.684 and ANN score = 0.911) is eliminated, the value increases up to 0.9922.

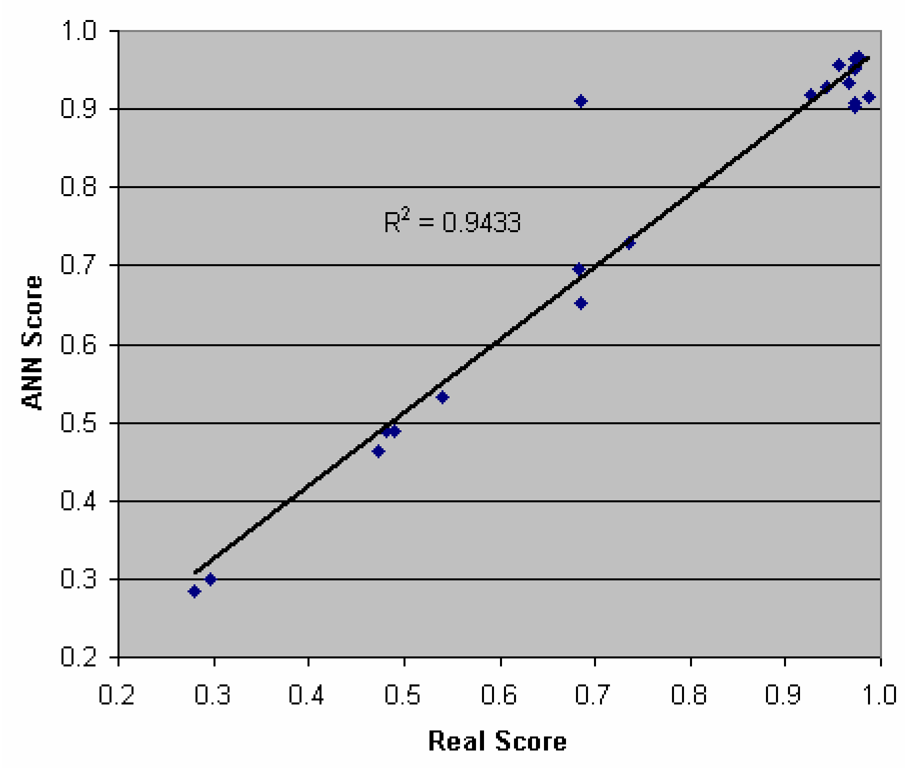

Figure 2. Relationship between Real Score and ANN Score

The result of ANN model was compared with that of multiple linear regression (MLR) model. When the data was processed by using statistical package SAS, 13 coefficient values and one intercept value were obtained as in Figure 3. Figure 4 shows the comparison between real condition score and MLR condition score. Because the R square value (determination coefficient) in MLR model was 0.9111, the ANN model appears to fit computed score into real score more efficiently.

In order to test ANN model's interpolation and extrapolation abilities, the 21 records were divided into two groups. Odd number records were used for ANN training; and even number records were used for verifying. When ANN model was trained with 11 records and verified with other 10 records, the determination coefficient was still high (0.8629) as shown in Figure 5. However, the MLR model obtained poor determination coefficient (0.0037) when the similar procedure was performed as shown in Figure 6.

Parameter Estimates

Variable	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|
Intercept	B	0.05633	0.51130	0.11	0.9147
x1	B	0.19455	0.30608	0.64	0.5408
x2	B	0.40038	0.30694	1.31	0.2231
x3	0	0	.	.	.
x4	1	0.06809	0.31869	0.21	0.8356
x5	1	-0.17396	0.14908	-1.17	0.2732
x6	1	0.41771	0.18329	2.28	0.0486
x7	0	0	.	.	.
x8	1	0.32192	0.18987	1.70	0.1242
x9	1	-0.08397	0.08152	-1.03	0.3299
x10	1	-0.01542	0.25450	-0.06	0.9530
x11	1	0.27704	0.17465	1.59	0.1471
x12	1	0.15736	0.07328	2.15	0.0603
x13	1	0.04565	0.09045	0.50	0.6259

Figure 3. SAS Results from Multiple Linear Regression

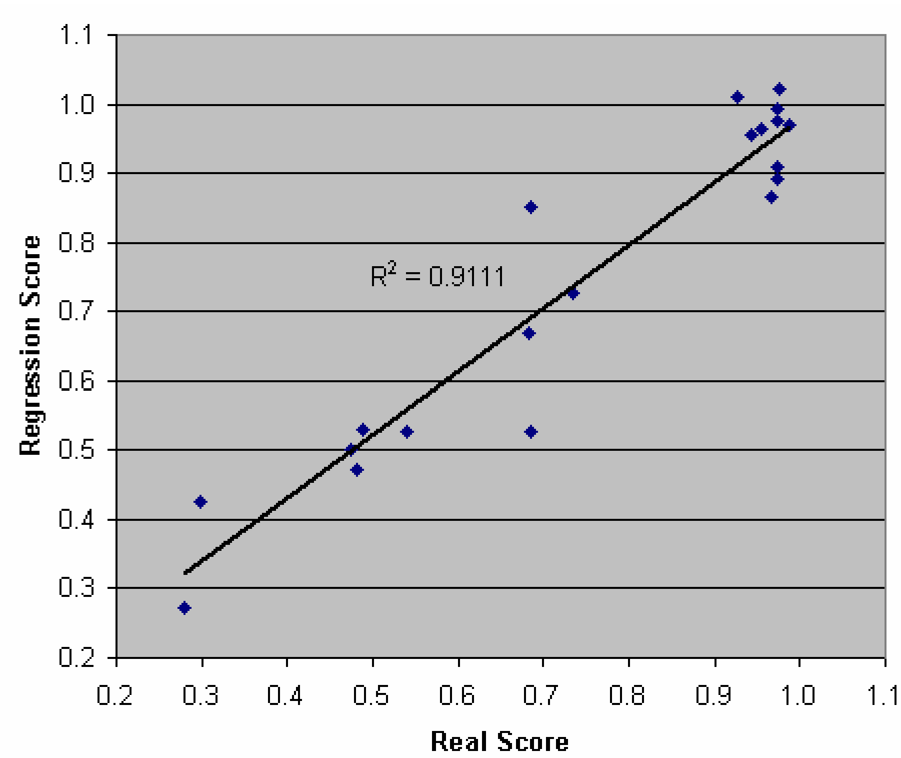

Figure 4. Relationship between Real Score and MLR Score

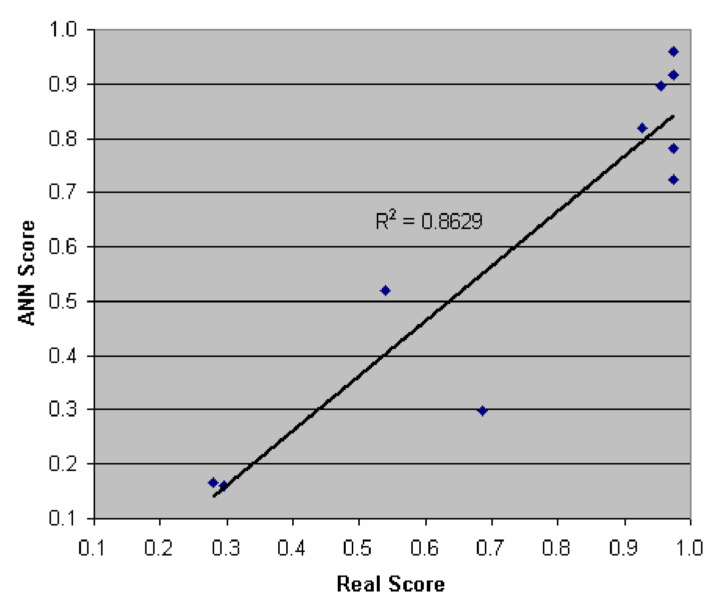

Figure 5. ANN Model's R Square for New Data

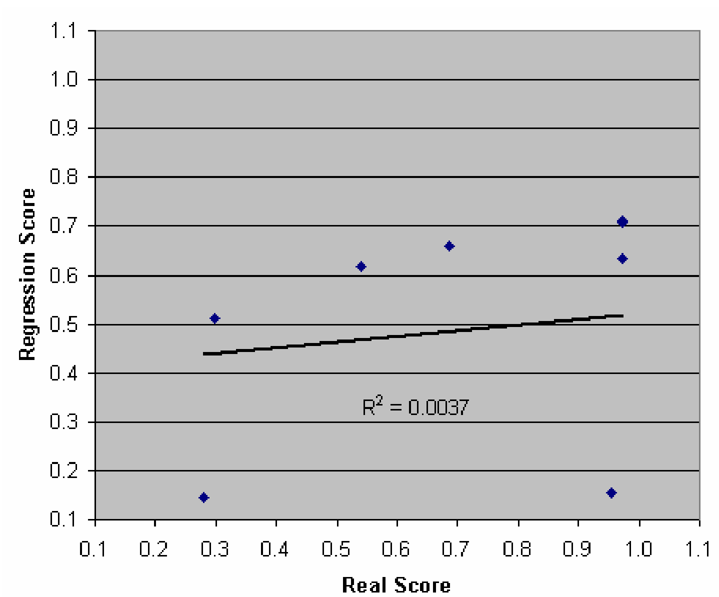

Figure 6. MLR Model's R Square for New Data

Conclusions

An ANN model was developed for the water pipe condition prediction without trenching ground. The model was then successfully applied to the representation of current pipe condition and the prediction of new pipe condition, when compared with MLR model.

The ANN model consisted of multi-layer perceptron structure and error back-propagation structure. For the MLP structure, one bias variable was used in input layer; and for the back-propagation structure, momentum technique was adopted to speed up the process. Also, a technique to consider more-than-two values for an input variable (pipe material) which cannot be quantified was proposed.

The input data of 11 fields and 21 records were prepared out of the measure raw data of over 50 fields and 61 records. The output data (overall condition index as a score between 0 to 1) was derived by considering five deterioration factors.

When the ANN model inputted current 21 records, it represented them with higher statistical correlation (R^2 = 0.9433) while the MLR model did with R^2 = 0.9111. Furthermore, when the ANN model trained with 11 records and verified with the rest 10 records, it still had quite high R^2 (0.8629) while the MLR model had very poor R^2 (0.0037). It should be noted that the ANN model proposed in this study can consider the high nonlinearity which the pipe data possesses whereas the MLR model fail to consider the nonlinearity in real-world data with respect to interpolation or extrapolation.

The ANN model in this study was implemented by using MS Excel VBA in order to enhance the user-friendliness. The author hopes many engineers in practice to use this model without any difficulty. For the future study, more pipe data will increase the usefulness of the model. And, instead of back-propagation technique, meta-heuristic techniques such as harmony search (Geem et al., 2001) can be used because it already performed better than back-propagation on benchmarking XOR classification problem (Geem et al., 2002).

References

ASCE Task Committee on Application of Artificial Neural Networks in Hydrology. (2000). “Artificial Neural Networks in Hydrology. I: Preliminary Concepts." *Journal of Hydrologic Engineering*, ASCE, 5(2), 115-123.

Dandy, G. C., and Engelhardt, M. (2001). “Optimal Scheduling of Water Pipe Replacement Using Genetic Algorithms.” *Journal of Water Resources Planning and Management*, ASCE, 127(4), 214-223.

Geem, Z. W., "Window-Based Decision Support System for the Water Pipe Condition Assessment using Artificial Neural Network," Proceedings of 2003 Conference of the Environmental and Water Resources Institute, ASCE, 2027-2032, Philadelphia, PA, June 2003.

Geem, Z. W., Kim, J. H., and Loganathan, G. V. (2001) “A New Heuristic Optimization Algorithm: Harmony Search.” *Simulation*, 76(2), 60-68.

Geem, Z. W., Kim, J. H., and Loganathan, G. V., "Application of Harmony Search Algorithm to Water Resources Problems," *Proceedings* of 2002 Conference of the Environmental and Water Resources Institute, ASCE, CD-ROM, Roanoke, VA, May 2002.

Gibbs, M. S., Morgan, N., Maier, H. R., Dandy, G. C., Nixon, J. B., and Holmes, M. (2006). "Investigation into the Relationship between Chlorine Decay and Water Distribution Parameters using Data Driven Methods." *Mathematical and Computer Modelling*, 44(5-6), 485-498.

Jain, A. (2002). "Short-term water demand forecast modeling techniques—Conventional methods versus AI." *American Water Works Association*, 94(7), 64–72.

Lee, C. Y., Shin, H. S., Kim, J. H., and Geem, Z. W., "Development of Deterioration Prediction Model for Water Distribution Systems using Probabilistic Neural Network (PNN)," *Proceedings* of the Conference of the Korean Water Resources Association, 499-504, Muju, Korea, May 1998.

Loganathan, G. V., Park, S., and Sherali, H. D. (2002). "Threshold Break Rate for Pipeline Replacement in Water Distribution Systems." *Journal of Water Resources Planning and Management*, ASCE, 128(4), 271-279.

Oh, C. S. (2000). *Introduction to the Neurocomputer*. Naeha Publishing, Seoul, Korea.

Rodriguez, M. J., Milot, J., Serodes, J. B. (2003). "Predicting trihalomethane formation in chlorinated waters using multivariate regression and neural networks." *Journal of Water Supply Research and Technology—Aqua*, 52 (3), 199–215.

Research on Safety Evaluation Model of the Main Underground Pipelines in Shanghai, China

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Abstract

With the rapid development of the economy in Shanghai—the metropolis in China, the density of the underground pipelines and the load of the pipeline network are increasing at a high rate. To guarantee the operation of the city properly, safety of Underground Pipeline Network (UPN) must be one of the foremost considerations. In this paper, the authors present a method to assess the UPN safety in Shanghai by applying a newly designed Safety Grade Evaluation Model (SGEM). Furthermore, this paper provides a safety management system, which will optimize management of UPN, by applying a Rehabilitation Cost Model (RCM). At last, the authors design a roadmap for SGEM for future applications. The above models enhance efficient management of pipeline network in Shanghai which will be helpful to rapid development of the city.

Keywords

Underground pipeline network, evaluation model, pipeline rehabilitation, safety management

Introduction

Shanghai, China's economic center, is playing an important role in Chinese economy. The city has 13.5 million residents[1], and lies in the east part of China; it is also called the locomotive of Yangtze Delta Area which makes a great contribution to Chinese economy. The gross domestic products (GDP) growth of Shanghai is shown in Table 1. Figure 1 shows the rapid development of Pudong

[1] This is the total resident number at the end of 2005.

District. Figure 2 shows the magnetic levitation train, the only line of its kind launched into business operation in the world.

Table 1. Shanghai economic growth ($1=7.8 RMB)[2]

year	1990	2000	2003	2004	2005	2006
GDP/Billion RMB	75.645	455.115	625.081	745.027	914.395	1029.697

Figure 1. Pudong in Shanghai

Figure 2. Magnetic levitation train

The rapid development of Shanghai's economy requires a high quality underground pipeline networks system. The most important function of the Underground Pipeline Network (UPN) is to provide major public utilities, and act as the lifeline of the city in case of emergency. Table 2 presents some statistics on growth of underground pipelines from 1990 to 2004.

Table 2. The growth of pipelines in Shanghai[3]

Indicator	Unit	1990	2000	2003	2004
Investment on infrastructure	Billion RMB	4.7	45	60.5	67
Length of gas and liquefied petroleum gas pipelines	kilometer	2,700	6,606	8,561	8,244.5
Length of water pipelines	kilometer	3,483	15,943	20,398	21,727

($1=7.8 RMB, 1 mile = 1.6 kilometer)

UPN Condition and Trenchless Development in Shanghai

Trenchless development status in Shanghai. The Shanghai's urban construction speed is much faster than other cities in China. Because of the development demands, Shanghai is one of the pioneer cities that adopt the trenchless construction methods. The statistics provided by SSTT (Shanghai Society for Trenchless Technology) indicates that, under support of Shanghai Government, the proportion of the underground pipelines construction projects which implemented by using trenchless technology is increasing from 0.2% in 1998 to 4% in 2002, and it is over 10% in 2004. In addition, the trenchless growth achieved 12% in 2006, and under reliable estimation, the figure will soar up to 24% in 2010.

[2] The statistic is from statistics datum of Shanghai Yearbook 2005.
[3] The data are from statistics of Shanghai Yearbook 2005.

The current maintenance pattern. When a pipeline failure happens, the government will execute a local repair on the broken section. Because of the long-term use and different designs of UPN in different districts, the underground pipelines vary from excellent to very poor condition. The current management pattern has some disadvantages as follows.

- The longer lag between the damage taking place on pipelines and repair has a heavy impact on the development of Shanghai.
- Rehabilitation benefit is so poor because the main pipeline lacks systematical plan more or less.
- The cost of management and rehabilitation of UPN is more than proactive safety management pattern which might be a systematic solution to the UPN.
- The existing pipeline safety management could not meet the demands of rapid development in Shanghai. The pipelines management department always works in tension status because the accident can take place in any time.

The development trend of the existing and future pipelines becomes more and more integrating. In the new developed Pudong District, the "irrigation, water supply, drainage" solution project has been chosen. In the long run, the development trend is to plan and design the underground pipelines system based on integration concept according to the partition of the administrative region. So the safety grade of UPN has great influences on economic development of Shanghai. Establishing safety evaluation system of UPN is a kind of advanced management pattern in the world. Figure 3 shows the new combined system in Pudong District. Figure 4 shows the Shanghai region on the map distinctly. The researchers select Shanghai as the study object for the following reasons.

Figure 3. New UPN system in Pudong[4]

Figure 4. Map of Shanghai region

Global Research Status of the Evaluation Model on UPN

Two German evaluation models. This paper presents two condition classification and evaluation models developed in Germany, which are excellent

[4] This project is designed by Shanghai Water Planning and Design Research Institute in Sep, 2003.

evaluation models that are applied widely in Germany. These two condition evaluation systems called ATV-M 149 and KAPRI have proved themselves over years of practices.

ATV-M 149 evaluation model is published in April 1999. This model presents a possible procedure for condition evaluation and classification that contains the condition classification and condition evaluation. The aim of establishing this model is to unify the different evaluation models in Germany. The main procedures include provisional condition classification, final condition classification, discovery of the evaluation points, discovery of the evaluation number and reclassification. The Evaluation Points provide the allocation into the condition class and can be calculated according to the Formula 1.

$$BP = CP + 100 \cdot Q \cdot H + 200 + 69 \cdot \left[INT \frac{(CP-1)}{100} - 1 \right] \tag{1}$$

Where: BP-Evaluation Points; CP-Condition Points; Q-Wastewater factor; H-Hydraulic factor; INT-Integer function, a factor that eliminates the decimal place; Priority List-Sequence of the rehabilitation needs.

The Evaluation Number (Formula 2) classifies the drain and sewer systems according to their condition class of the type of sewer, the affected interests protected by standards or law, as well as the evaluation points.

$$BZ = ZK_f \cdot 10^5 + KA_f \cdot 10^4 + SR_f \cdot 10^3 + BP \tag{2}$$

Where: BZ-Evaluation number; ZK_f-Condition class factor; KA_f-Sewer type factor; SR_f-Interests protected by standards or law; BP-Evaluation point (if the evaluation points cannot be found, the condition points CP can be used in their place). The priority list consists of the downgrade evaluation number. Its provides a sequence for the measures to be implemented in the sequence of the condition classed of the type of sewer, the interests protected by standards or law as well as the evaluation or condition points.

A mature and excellent classification system that has long been used in Germany is the KAPRI system. The basic concept of this model is to frame priority list for the rehabilitation of individual points of damage and of sections of the sewer by means of pure statistical damage classification. For this purpose, a selected sewer section to be evaluated will take structural condition and external limiting conditions into account by using corresponding technical means. Then, the complete condition evaluation of the selected sewer section and the basis for the priority lists created are gained by the mathematical linking of the results found respectively. Two main procedures called evaluating the structure condition and evaluating the limiting consist of the KAPRI condition evaluation system. Then after the complete condition evaluation process the final step is condition classes, and the priority list is created according to the results.

The Safety Grade Evaluation Model (SGEM) Based on New Concept

Establishing SGEM for Shanghai UPN has the advantages as follows.

- Take initiative on safety grade evaluation of UPN in China which could provide references to other cities.
- Ensure the safety, high efficiency and standardization of UPN management system.
- Accelerate and enlarge the development of trenchless rehabilitation market and regulate administrative regulation system.
- Boost the development speed of urban construction.
- High safety and reliability concept of underground space design and management will make contribution to national security and economy which are influenced by special type pipelines that play an important role in UPN.

Aiming at the safety management of UPN, this research plan chooses a kind of safety grade evaluation model which include more consideration about full-scale effect of the society. The other pipeline evaluation models are inclined to consider the cost of pipeline rehabilitation and management more. The SGEM includes the following 3 parts.

Condition Grade (CG). The condition grade of pipeline is a basic indicator of SGEM which has been researched by the developed countries perfectly; the available ATV-M 149 and KAPRI could help to frame the condition evaluation method suitable for Shanghai. CG indicates the actual condition of pipeline, so the damage information is synthesized in this factor.

Plan and Design Grade (PDG). PDG consists of Seismic Fortifying Grade (SFG), Operation Grade (OG), Management Grade (MG), Design Grade (DG), and Fire Protection Grade of Structure (FPGS) nearby. The PDG is an indicator that expresses the safety consideration based on various factors which consist of seismic safety, operation safety, management safety, design safety and the fire protection safety. The fire protection grade means that the water pipelines might affect safety of the buildings nearby. Take an extreme case for example, if the shopping plaza takes fire, but the water supply system damages, which take no effect on putting out the fire, so it could become a severe disaster. Moreover, the fire protection grade also indicates the importance of the buildings adjacent to the pipelines.

Impact Grade (IG). The economic loss is the best embodiment of the impacts caused by of UPN safety problem. IG is expressed mainly by the economic factor. IG includes Direct Impact (DI) and Indirect Impact (II). Direct impact includes Political Impact (PI), the Direct Economic Loss Impact (DELI), Psychological Impact (PLI), and Living Impact on Residents (LIR). DELI consists of the

Economic Loss Impact of Equipment (ELIE) and Economic Loss Impact of UPN Structure (ELIS). Indirect impact comprises four aspects which are the economic ripple effects caused by the emergence and disaster of UPN. The Indirect Impact is explained as follows.

- The Economic Loss Impact on Stop Production or Reduce Production (ELISR) of the industry caused by safety problem of UPN.
- The Economic Loss Impact on the Benefit produced among industrial branches (ELIB).
- Impact of Damage on Rehabilitation Cost (IDRC).
- Impact on Urban Investment Benefit (IUIB).

Figure 5 shows the Structure diaphragm of SGEM which includes all the factors considered by researchers. The SGEM has lots of functions as follows.

- All above factors are considered by this safety evaluation model, which is a new evaluation system based on new safety concept
- The calculation outcome of the model could be used for security agency for evaluating the importance and influence of the UPN safety to the security of nation
- The priority list calculated by the SGEM is also effective which emphasizes more consideration on safety of UPN. For example, pipe A that is calculated by the normal condition evaluation model could run about 30 years, but it must be rehabilitated at the end of 25 years operation for the security reasons if selecting the SGEM. Although the rehabilitation work is ahead of the normal schedule, the society gets the more security insurance.

The safety coefficient calculated by SGEM could be used for the cases which consider more on safety factor or the safety is most important factor to the pipeline. For example, the pipeline used by military or having strong relationship with the nation security may choose this model. To sum up, the SGEM is a kind of evaluation model which synthesizes more factors especially the safety. Surrounded by many economic evaluation models, SGEM has its own advantage to develop perfectly with great space. Table 3 shows the safety evaluation process of pipe k, Table 4 shows the evaluation of PDG on pipe k, and Table 5 shows the evaluation of IG on pipe k.

Table 3. Safety evaluation of pipe k

The safety grade of pipe k			
CG	PDG	IG	$\sum$
$G_{(safety,\ cg),k}$	$G_{(safety,\ pdg),k}$	$G_{(safety,\ ig),k}$	$G_{(safety,\ k)}$
$G_{(safety,\ k)} = (CG+PDG+IG)_k$			

Table 4. Evaluation of PDG on pipe k

The plan and design grade of pipe k					
SFG	OG	MG	DG	FPGS	$\sum$
$G_{(pdg, sfg),k}$	$G_{(pdg, og),k}$	$G_{(pdg, mg),k}$	$G_{(pdg, dg),k}$	$G_{(pdg, fpgs),k}$	$G_{(safety, pdg),k}$
$G_{(safety, pdg),k} = (SFG+OG+MG+DG+FPGS)_k$					

Table 5. Evaluation of IG on pipe k

The impact grade of pipe k											
DI						II					$\sum$
LIR	PI	ELIE	ELIS	PLI	$\sum$	ELISR	ELIB	IDRC	IUIB	$\sum$	
$G_{(di, lir),k}$	$G_{(di, pi),k}$	$G_{(di, elie),k}$	$G_{(di, elis),k}$	$G_{(di, pli),k}$	$G_{(ig, di),k}$	$G_{(ii, elisr),k}$	$G_{(ii, elib),k}$	$G_{(ii, idrc),k}$	$G_{(ii, iuib),k}$	$G_{(ig, ii),k}$	$G_{(safety, ig),k}$
$G_{(ig, di),k} = (LIR+PI+ELIE+ELIS+PLI)_k$						$G_{(safety, ig),k} = (ELISR+ELIB+IDRC+IUIB)_k$					
$G_{(safety, ig),k} = (DI+II)_k$											

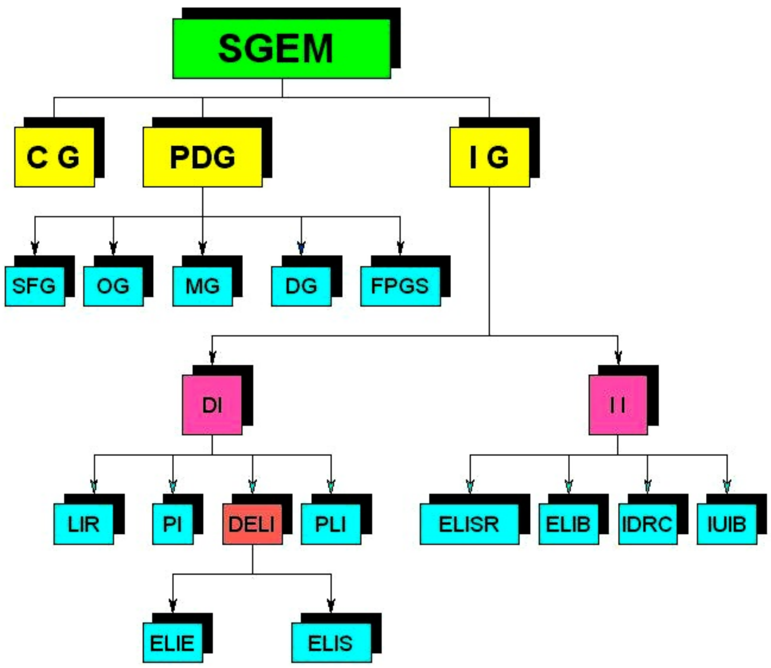

Figure 5. The SGEM structure

The Significance of SGEM

Currently the development and the management of urban underground space, one of the major parts in the city, have a strategic impact on the future status and economic development roadmap of Shanghai. The ultimate meaning of SGEM is to optimize the management of the urban underground space, improve the benefit of urban development and accelerate the economic growth in the city.

The increasing urban construction results in the tension of future land use. Skyscrapers on the ground are one of the development trends of city. There is rapid development of the pipelines and various constructions such as the metro, underground pass way, and the vehicle tunnel to pass the river in underground space. For special reasons, many cities have restrictions on building height, so the trend of development to underground is enhanced. If there is a clear recognition on the safety condition and grade of urban UPN, it might be helpful to the development quality of underground space. The safety grades of UPN, which are evaluated by the SGEM, could be consulted by the relative department to make a safety management and scientific development of UPN. Application of safety evaluation grade which is the purpose of establishing the model, could take an example as follows.

For the pipeline that evaluated as a high safety grade, the management department and the design institute could select corresponding grade of design, fortifying, rehabilitation and management to create the most safety benefit from UPN. Besides, the high safety grade pipeline also chooses the corresponding proactive treatment grade, if it is damaged after disaster, the requirement of rehabilitation time and safeguard of re-operation are stricter than that of the low grade pipelines. The meaning of safety evaluation on UPN is providing the reference factors which will be helpful to urban development and management of underground space directly and scientifically.

Rehabilitation Cost Model (RCM) Based on SGEM

Combined with the discussion on the SGEM, one of the SGEM major functions is to frame the priority list which is based on safety consideration. From the priority list, the maintenance period could be checked easily. The construction cost combined with the safety factor of pipeline rehabilitation is also calculated by the rehabilitation cost model based on SGEM. RCM could be used for the management department to make decision and select the suitable rehabilitation methods. The economic application of RCM (Formula 3) includes many aspects as follows.

- Value the cost of rehabilitation methods based on safety management
- Choose the suitable rehabilitation method for the priority list based on safety management
- Contrast the bid programs and choose the suitable one based on safety management
- Estimate the proportion of safety management cost on one pipeline rehabilitation project
- Promote the R&D of new trenchless rehabilitation methods based on safety management

$$Z_{(safety,m),\hat{K}} = \sum_{k=1}^{K}\left[\alpha_{(safety,m),k} \cdot G_{(safety,k)} \cdot C_{(economy,m),k}\right] \qquad (3)$$

Where: $Z_{(safety,m),\hat{K}}$ -Total cost over the whole pipelines that using rehabilitation methods *m* based on safety management; $\alpha_{(safety,m),k}$ -Economic compensatory coefficient of pipe *k* based on safety management when the rehabilitation means *m* is selected; $G_{(safety,k)}$ -Safety grade of pipe *k* based on SGEM; $C_{(economy,m),k}$ -Rehabilitation cost of pipe *k* using rehabilitation method *m* based on normal economic benefit only.

The RCM is based on SGEM abandons the design idea that only takes the economic factor into account. RCM is combined with the economic and safety factors organically to develop a new concept of economic evaluation method which is mainly used in pipelines rehabilitation field for safety management that benefits the city. Safety management structure of Shanghai UPN is shown in Figure 6. The RCM based on SGEM has many advantages as follows.

- The new model that thinks over all factors referred by SGEM and the price calculated by the model is comprehensive including the safety consideration which is added in the cost of project
- The model could be used by the national security department which evaluates the cost of fatal pipeline exactly and for the most important pipelines government could appropriate the money for construction reasonably
- Since taking all of the factors into account, the result may be different from other methods, especially the great difference on cost may occur, but these are all reasonable and correct
- The model is a framework comprised the new concept of safety management which aims at maximum of the cost and safety on pipelines

Application Roadmap of SGEM in Shanghai

Taking Shanghai as a case study, Pudong District will be the first district to apply the effectiveness of the evaluation, then, it will be used in Shanghai after permitted by the Shanghai government. The permit of using this system all over the country depends on the success of new safety management pattern theory and high credibility of initial basic database about UPN in Shanghai. If Shanghai takes more benefit from this pattern, it will popularize in all of the cities in China after permitted by the Ministry of Construction P.R.China. This research, which provides an all-round reference for plan-design and safety management of urban development in China, has great potential on development. The government has the responsibility and obligation to support this research. Application roadmap is shown in Figure 7.

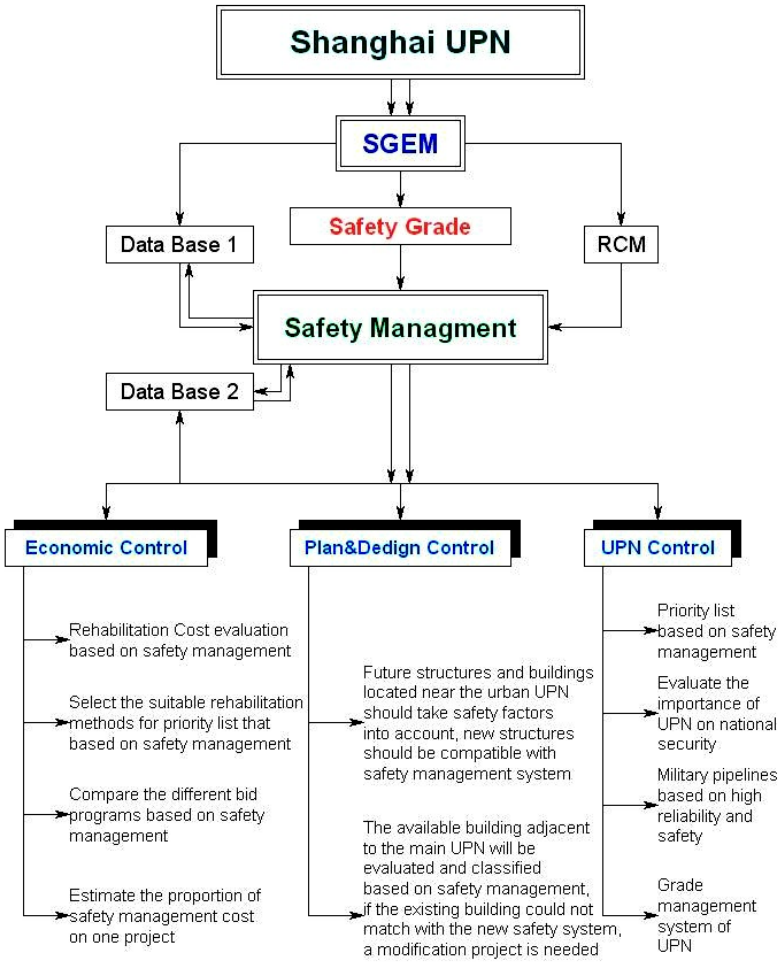

Figure 6. Safety management structure of Shanghai UPN

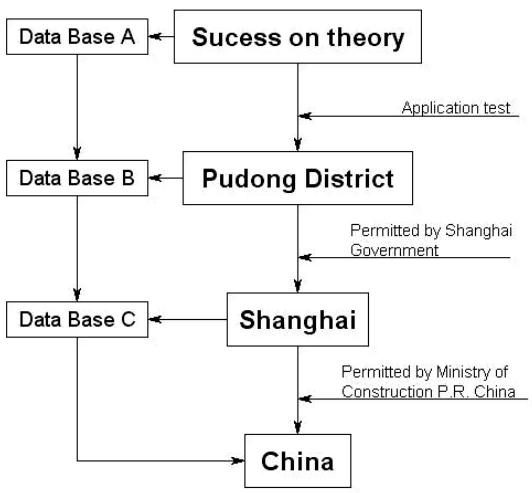

Figure 7. Application roadmap of SGEM in China

Conclusions

This paper presented the following:

- A new concept for underground pipelines safety management
- The SGEM for optimizing the underground pipeline network.
- The RCM based on SGEM is also established initially to evaluate the rehabilitation cost that synthesizes the safety and economic factors.
- A framework for implementation of a safety management strategy.

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List of Abbreviations

Symbol	Definition	Symbol	Definition
BP	Evaluation Points	*II*	Indirect Impact
BZ	Evaluation number	*IUIB*	Impact on Urban Investment Benefit
CARE-S	Computer Aided Rehabilitation of Sewer Networks	KA_f	Sewer type factor
CARE-W	Computer Aided Rehabilitation of Water Networks	*LIR*	Living Impact on Residents
CG	Condition Grade	*MG*	Management Grade
CP	Condition Points	*OG*	Operation Grade
DELI	Direct Economic Loss Impact	*PDG*	Plan and Design Grade
DG	Design Grade	*PI*	Political Impact
DI	Direct Impact	*PLI*	Psychological Impact
ELIE	Economic Loss Impact of Equipment	*RCM*	Rehabilitation Cost Model
ELIS	Economic Loss Impact of UPN Structure	*SFG*	Seismic Fortifying Grade
ELIB	Economic Loss Impact on the Benefit	*SGEM*	Safety Grade Evaluation Model
ELISR	Economic Loss Impact on Stop Production or Reduce Production	SR_f	Interests protected by standards or law

FPGS	Fire Protection Grade of Structure	*UPN*	Underground Pipeline Network
IDRC	Impact of Damage on Rehabilitation Cost	ZK_f	Condition class factor
IG	Impact Grade		

Bibliography

Baur, R. (2004). "Long term strategies for network rehabilitation and their implementation into annual plans." *No-Dig 2004 conferences proceedings*.

Binsbergen, A.V. and Visser, J. (2001). Innovation Steps towards Efficient Goods Distribution Systems for Urban Area, *DUP Science*, The Netherlands, 97-103 & 265-272.

Li, J. (2005). Structural Response Network Disaster Simulation Lifeline Ground Motion Reliability, *Science Press*, Beijing, 51-66

Liu, C.G. (2005). Introduction to Lifeline Earthquake Engineering, *Dalian University of Technology Press*, Dalian, 362-370&287-291.

Mertova, J. (2004). "Assessment of Multi-critical Sewer Network Reliability for Risk Analysis." *No-Dig 2004 conferences proceedings*.

Najafi, M. and Kulandaivel, G. (2005). "Pipeline condition prediction using neural network models," *Proceedings* of ASCE Pipeline Conference, August 21-25, Houston, Texas.

Olliff, J. and Rolfe, S. (2002). "Condition Assessment: The Essential Basis for Best Rehabilitation Practice." *No-Dig 2002 conferences proceedings*.

Sagrov, S., and Schilling, W. (2002). "Computer Aided Rehabilitation of Water Networks (CARE-W)." *No-Dig 2002 conferences proceedings*.

Sagrov, S., and Schilling, W. (2004). "Strategic Rehabilitation of Water Distribution and Wastewater Collection Systems." *No-Dig 2004 conferences proceedings*.

Silva, D.D., and Davis, P. (2002). "Condition Assessment of Cast Iron and Asbestos Cement Pipes by In-Pipe Probes and Selective Sampling for Estimation of Remaining Life." *No-Dig 2002 conferences proceedings*.

Sinha, S.K. (2003). "Development of Prototype Municipal Asset Management System." *No-Dig 2003 conferences proceedings*.

Stein, D. (2001). Rehabilitation and Maintenance of Drains and Sewers, *Ernst & Sohn*, Berlin, 288-301.

Stein, R., and Alvarez, R.T. (2004). "'STATUS Sewer' A holistic model for efficient net management based on section specific aging prognoses." *No-Dig 2004 conferences proceedings*.

Webb, R.E., and Ariaratnam, S.T. (2003). "Technologies for the Assessment of Large Diameter Lined Concrete Sewers in the City of Phoenix." *No-Dig 2003 conferences proceedings*.

Wessels, J.W. (2003). "Condition Assessment of Sewers for Replacement/Rehabilitation and The Accompanying Development of a Management System." *No-Dig 2003 conferences proceedings*.

102-Inch Cliff Pipe Rehabilitation

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Abstract

After the 4.5-mile (7.2 km) , 50-year-old, 2.6 m (102-inch) Olmsted Flowline Pipe was acquired from a power company, Central Utah Water Conservancy District (CUWCD) investigated ways to increase its reliability and reduce its maintenance. Its lower "Reach A", a 0.6 km (2000-foot) reach of pipe on a cliff, was failing due to punctures from boulder falls, steel trestle failures, concrete support cracking, pipe lining and coating failure, and inadequate maintenance access.

CUWCD rehabilitated this pipe on a cliff in order to provide a spillway for a new regulating reservoir, provide hydropower reliability, reduce operations and maintenance (O&M) in a hazardous work area, and improve canyon aesthetics. CH2M HILL designed rehabilitated facilities for relocating the pipe (to "tuck" it into the cliff), textured concrete surfacing to aesthetically blend pipe encasements into the cliff, a drivable access for the cliff pipe, rock anchors to secure the pipe to the cliff, and vegetated reinforced earth slopes at non-cliff reaches. The Olmsted System spillway for a new 10 million gallon (MG) tank were maintained by the cliff pipe rehabilitation where flows were split between the hydropower system and CUWCD's municipal and industrial (M&I) system.

Steep earthen cross slopes were soil nailed to allow for a buried pipe replacement of the Flowline. This approach eliminated talus buildup on an above ground pipe while stabilizing slopes uphill of trench cuts so they would not fail into the new buried pipeline in construction or in perpetuity.

Background

The 4.5-mile (7.2 km) Olmsted Flowline was first built in 1912 as a wooden flume (see Figure 1 below) on trestles to convey water to a hydro electric plant at the mouth of Provo Canyon. In its lower 0.6 km, the flume traversed a steep limestone cliff and then connected to a rock tunnel where a window in the side of the cliff was developed to spill flows back to the river when turbines from the hydroelectric plant shut off.

FIGURE 1
The left photo shows the flume cross section. The right photo shows the flume upstream of the cliff reach.

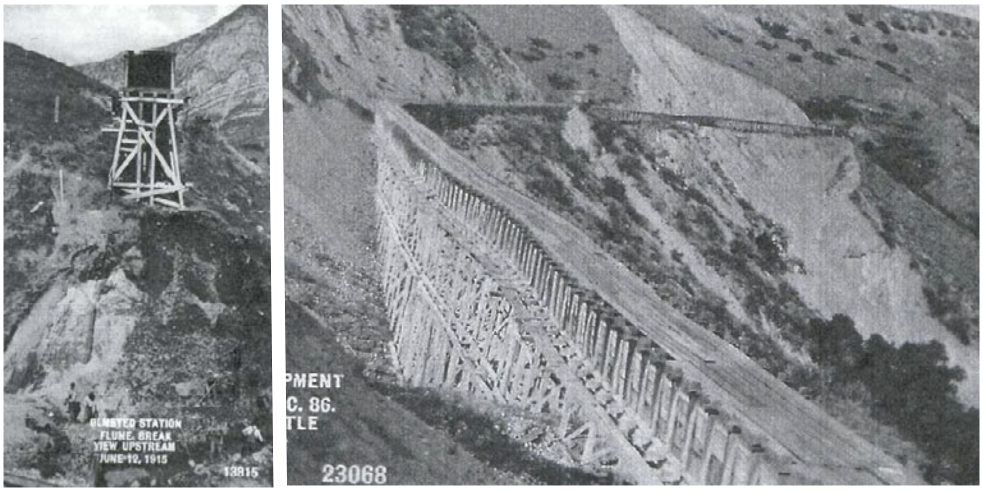


FIGURE 2
1914 photo of the cliff reach flume and trestle.

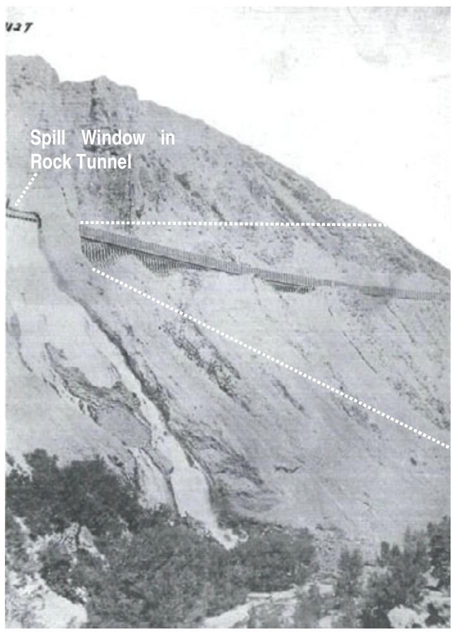


FIGURE 3
Olmsted flume at transition to rock tunnel.

Figure 2 shows the cliff reach of the flume. At the upper left, water can be seen spilling out of the window in the rock tunnel. Figure 3 shows a man standing by the trestle and flume in the cliff reach just before it connects to rock tunnel.

FIGURE 4
1999 photo of the cliff reach pipe and supports. Note the overflow is now on supports upstream of the rock tunnel.

In 1948 and 1952, the wooden flume was replaced with a thin-shell 5 mm (3/16 inch), above-ground 2.6 m (102-inch) diameter steel pipe. Fifty years later, the 102-inch pipe was still operating (seen in Figure 4 at the cliff reach), though with numerous reliability concerns. In the 1950 reconstruction, the overflow was moved 200 feet upstream of the rock tunnel and was redesigned to function as a siphon to accelerate discharge of excess water if the free water surface in the overflow chamber exceeded its design gradient.

In 1987, the United States Bureau of Reclamation (USBR) purchased the Olmsted pipeline and facilities from the power company to prioritize its functions to meet M&I water demands. During acquisition, it was known the Flowline would require substantial rehabilitation to meet the M&I reliability needs of Utah's larger population centers.

Since CUWCD had the most vital M&I water supply interests, they were given responsibility for its O&M, rehabilitation, and/or replacement. After replacing the Flowline at a fast-moving landslide (in 1989) and at its river diversion (in 1995), CUWCD addressed the remaining Flowline needs through a Capital Improvement Plan (CIP) in 1997. The plan addressed capacity constraints, rock falls, talus buildup, structural failures, snow avalanches, corrosion, landslides, and other issues including the Flowline's rehabilitation on a cliff reach and its replacement and on several steep cross slopes.

Figure 5 shows the lower 3.3 km (2 miles) of the Flowline within Provo Canyon with the old 102-inch aboveground pipe still in place. The cliff reach where the existing pipe was rehabilitated is in the background. A flow regulatory station and reservoir were added just upstream of the cliff to regulate flows to the cliff reach and M&I system. Upstream of the reservoir are several steep cross slope reaches where talus slope problems (discussed later) were addressed using soil nails to stabilize slopes and old pipes were replaced with new buried pipes. Reaches with creeping landslides (addressed in earlier papers) were improved through above ground replacement pipes.

FIGURE 5
1998 photo of the lower 3.2 km (2 miles) of Provo Canyon with pipe reliability issues and solutions noted.

Value Engineering and Design

In 1999, CUWCD held a CIP value planning session and then began preliminary design in 2000. After preliminary design and value engineering, CUWCD adopted the combined CIP, value planning/engineering, and preliminary design recommendations as follows:

- Rehabilitate 1.7 km (1.0 mile) of above- and below-grade 3.1 to 2.6 m (120-to 102-inch) steel and RCP pipes, including a 0.9 km (0.5 mile) reach on a cliff
- Replace 4.2 km (2.5 miles) with new, larger, buried pipe in existing easement
- Stabilize slope cuts with soil nails to protect new buried pipe for 1.2 km (0.8 mile)
- Cross two creeping landslides (5mm/yr or 0.1 in/yr) with new above ground pipe
- Add a flow regulating station and reservoir to increase system operating flexibility

This paper addresses issues within the adopted first and third bullet recommendations.

Cliff Pipe Rehabilitation

Hydropower and Spill Functions Dictated Rehabilitation in Place

The cliff pipe had two essential functions: (1) to provide a spillway path to the river for system overflows, and (2) to deliver water to a hydroplant. Although the CIP considered alternative spillway paths, the existing spill easement on the cliff and its right to spill up to 450 cubic feet per second (cfs) abruptly into the river made rehabilitating the pipe on the cliff worth the expensive construction. CUWCD's contractual obligation to deliver water to the hydroplant solidified their decision to maintain a dual-function conduit on the cliff to address the historic problems from rock falls, failing supports, talus buildup on pipe, and poor access to address frequent pipe repairs.

FIGURE 6
Pipe relocated into the cliff. Note earth filled fault zone in background.

Use of Existing Pipe as Concrete Form

The CIP and value planning recommended to preserve a 450 cfs conduit to the cliff spillway easement and to concrete encase and rock bolt the pipe on the cliff to address the historical problems and provide driving access onto the cliff.

Allow Box Culvert as Alternative

Because the cliff pipe operated with a free water surface, the value engineering review team suggested allowing contractors build a box culvert as an alternative conduit to concrete-encasing the existing pipe. This option was allowed but was not chosen by the contractor, Gerber Construction.

Relocate Pipe into Cliff

To reduce concrete mass and make the encased pipe less visible in the canyon, the design showed the pipe being relocated into the steep rock slope in two locations. Although this required blasting into the hard limestone rock, the blasting reduced the concrete mass and increased the concrete to rock bonding surface area and the strength of the native rock to which the encasement would be bolted. In the end, the contractor requested, and was allowed to move the pipe further into the cliff than the design required. Figure 6 shows clean rock subgrade before placing concrete where pipe was relocated into slope.

Encasements Span Earthen Fault Zone

Another reason to relocate into the cliff was to increase the rock support at both ends of a 20-foot-wide earth-filled fissure in the cliff (seen at top of Figure 6). The fissure was an inactive fault that had filled with soil. Additional rock bolts were used on both sides of the fissure to support the encasement on the rock and not on the erodible soils.

Lighten Mass Concrete at Cliff Recesses

On the pipe's north side (cliff side) for 400 feet before it entered a rock tunnel there were two 100-foot long cliff recesses (where the cliff face was up to 50 feet away from the straight pipe). Between these recesses was an 80-foot-wide and hundreds of feet tall rock tower which was abutting (within 2 feet of) the pipe. The rock tower was deemed stable and too massive to remove. It was decided to encase this reach of pipe in place and concrete fill the recesses north of the pipe so that rock falls and ice would not accumulate behind the encased pipe. Such a large mass of solid concrete would require extensive rock bolting, so the rock bolt designer, Gerhart Consultants, recommended an elevated cast in place concrete slab (at the top of pipe encasement) with large empty rooms beneath the slab. These rooms were divided by shear walls that were bolted to the cliff.

The concrete slab was designed to "out-slope" toward the top of pipe encasements so accumulations of rock fall materials on the slab could periodically be bladed across the top of the encasement and pushed off down the cliff. Gerber Construction requested, and was allowed, to fill these 15- to 25-foot wide rooms with closed-cell polystyrene rather than abandoning forming inside the inaccessible rooms. Altogether there were about nine rooms that were built this way, three in each cliff recess, and three under the pipe (see Figure 7).

FIGURE 7
Typical room under concrete pipe before it was filled with polystyrene foam blocks. Siphon spillway is at upper right.

Encase Existing Siphon Spillway

The existing 450 cfs siphon-type spillway had functioned well and was left in place and concrete encased within the new structure with a new reinforced concrete roof over the spillway entrance chamber (see Figure 7). Gerber Construction use curved walls and sloped walls around this structure to simplify forming (see left photo in Figure 8). This also improved aesthetics.

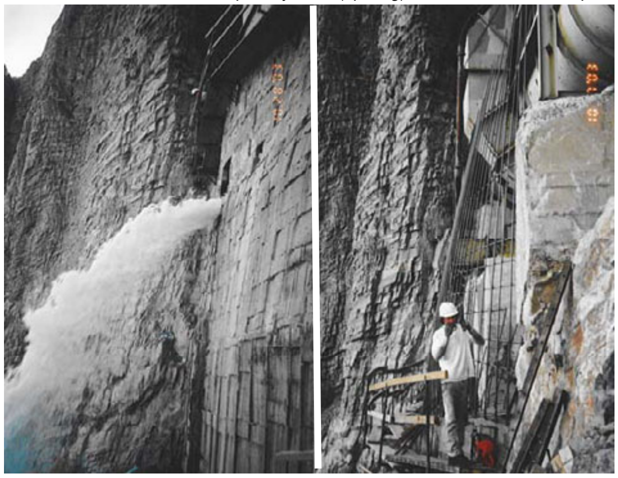
Textured Concrete Face

The design required textured concrete faces and test panels to demonstrate form liners for rock texturing would simulate native cliff face textures. Gerber successfully used polystyrene form liners cut with hot wires into jagged patterns to accomplish this (Figure 8 finished concrete facing).

Safety Railing

A heavy duty safety railing was provided atop the vertical face of the concrete encasement to protect persons or equipment while pushing rock fall materials over the encasement and down the cliff.

FIGURE 8
Textured concrete face at Spillway after (spilling) and before concrete placing.



Concrete Fill at Rock Tunnel and at Cliff Window

Where the existing pipe passed through about 80-feet of rock tunnel, the crown of the tunnel was raveling and large boulders falling onto the pipe. Loosened rock materials at the crown of the tunnel were removed and the concrete encasement was extended past the top of the pipe up to a stable rock crown in the rock tunnel. Fill concrete was also extended to 2 to 3 meters (6 to 9 feet) above the pipe to fill recess in the cliff created for the pre-1950 Flowline spill window (Figure 9).

FIGURE 9
Filling recess over cliff window.

Rock Doweling and Rock Bolting

The encasement anchoring design called for the native rock to be excavated for at least 2 feet of width below the vertical concrete facing of the new encasement wall. Footings at the base of this wall were required to have rock dowels. The rock dowels acted in shear to protect the large concrete masses from sliding down the steep grade.

Near horizontal rock bolts were designed to act in tension to secure concrete masses to the cliff. Bolts tied steel plates, anchored in the concrete encasements, to bolt lengths grouted 15 to 30 feet into limestone bedrock. Rock bolts were designed to be installed first and cast into concrete, but at contractor request, were placed after concrete encasements by placing steel sleeves and plates through the encasement forms and then drilling rock anchor holes through the sleeves. After encasing pipe, bolts were inserted, grouted pull-tested, and then tightened against steel plates (see Figures 10 to 13) with hex nuts.

FIGURE 10
Drilling rock bolts with sleeves through encasements.

FIGURE 11
Drilling rock bolt under pipe.

FIGURE 12
Installing rock bolt.

FIGURE 13
Pull-testing rock bolt on steel plate at face of textured wall.

Soil Nailing Stabilizes Uphill Slopes at Trench Cuts

Besides cliff reach encasements and rock bolting, a 2 mile reach of 120-inch replacement pipe traverses 1.6 km (1 mile) of steep cross-slopes with raveling talus rocks (see Figure 14) in an inactive ancient landslide. Buried pipeline construction in these areas would require large trench cuts parallel to contours which would weaken these slopes. To prevent construction-induced slope failures near the pipe, uphill trench slopes were permanently soil nailed and shotcreted to restore strength to slopes weakened by trenching.

FIGURE 14
Talus buildup on pipe.

Steep cross slopes and a narrow 6-m (20-foot) wide "bench," formed by the ancient Lake Bonneville shoreline, typify the native slopes over which the Flowline passes. The 1912 flume and 1952 aboveground 2.6 m (102-inch) pipe construction both widened this bench, cutting into the native 2:1 to 1.4:1 slopes above the bench and filling (mildly over-steepening) the slopes below the bench. This caused once stable slopes to ravel. Over the 50-year life of the aboveground steel pipe, over 8 feet of talus rock had accumulated on the uphill side of the pipe in several reaches (including the cliff pipe reach).

The decision of how to build a 3.05 m (120-inch) pipe parallel to contours on steep slopes involved a review of several variations of above- and below-ground pipe:

An above-ground pipe replacement option would have disturbed existing slopes little. Except for talus raveling, slopes had appeared stable for the 90 years of Flowline operation and through multiple hydrologic years that exceeded a normal 100-year wet cycle. However, the above-ground pipe option failed to address historical problems of talus buildup on the pipe (which moves the pipe off of its supports, see Figure 9), the pipe's high visibility in canyon, vandalism, expansion joint maintenance, and narrow access roads.

A buried pipe had none of these disadvantages and was lower cost. Yet cutting the slope to bury a 3 m (120-inch) pipe would weaken existing slopes above the pipe. Options to raise the ground (to bury the pipe) without deep cuts were evaluated, because these required large earth imports that would surcharge the slopes and encourage downhill slope failures.

After analysis, it was decided that balancing earthwork on the slope would keep slope weights near their native state and reduce slope destabilization. The "balanced earthwork" approach required trench cuts 2 to 3 m (7 to 10 feet) deeper into the slope than the 1912 and 1952 construction cuts. The cut-weakened slopes would be inadequately buttressed by the pipe (a 3 m [10 ft] "hole in the ground") and its backfill.

To restore and augment native slope strength, a soil nail wall was provided to reinforce the slope uphill of the pipe trench. Hollow-core corrugated steel soil nails were drilled and grouted into place until grout exited the ground around the nail hole. Nail depths were designed to fit native slope strength. Nail embedment depths were generally equal to the vertical height of the wall or 4.6 to 7.6 m [15 to 25 feet]) and sloped downward into slope at 15 degrees off horizontal.

Soil nail wall faces were started with a row of near-vertical soil nails, or "micropiles," spaced 1.2 m (4 ft) apart (spacing depended on local soil-bridging ability). After 5 to 8 feet of initial excavation was done below micropiles, the first soil nail wall lift was installed. Each soil nail wall lift included back-of-wall drain strips, weep hole pipes, reinforcing, shotcreting (150 mm [6 inches] thick) and curing; then drilling and testing a row of soil nails through the shotcrete, followed by plating and bolting of soil nail heads at the face of the shotcreted wall.

Subsequent lifts of excavation, soil-nailing, grouting, testing and shotcreting extended soil nail walls down to the base of the existing pipe. Extra soil nail lengths were trimmed off and a final shotcrete layer was placed to cover nail caps, plates, and nuts. Figures 14 to 17 show soil nail wall placement and testing (in the summer) when the 102-inch pipe was in operation.

After installing soil nail walls with the old pipe in operation (Figure 14), the old pipe was removed and trenching, placement (Figure 15), and backfill (Figure 16) of a new 120-inch pipe ensued over a 5-month winter shutdown.

FIGURES 15 TO 18

A soil nail wall starts with a row of vertical "micropiles" (Figure 15 at left) followed by a number of rows of soil nails, with multiple 6-foot high wall lifts of reinforcing, drains, and shotcrete (Figures 16 and 17 at center). Each nail is tested to verify it has the required pull-out strength (Figure 18 at right).

FIGURES 19 TO 21
A completed soil nail wall (Figure 19 at left) reinforces an undercut cross-slope to allow for placement (Figure 20 at center) and backfilling (Figure 21 at right) of a new buried 120-inch pipe.

Concrete Saddles and Encasements for Lateral Support

To allow working room on the narrow bench, the contractor was allowed to remove the downslope native earth that would laterally support the new pipe provided he rebuild it at 95 percent relative compaction. Concurrent with rebuilding this fill a cement slurry pipe zone bedding was placed in multiple lifts to 0.7 times the pipe diameter. Where benches were too narrow for a stable downslope fill, or where supporting slopes had questionable support value (such as where the old flume had failed and washed out native slopes and these earth slopes were rebuilt by manual construction methods), the new pipe was concrete encased to provide the needed lateral support.

Conclusion

The Olmsted Flowline Replacement/Rehabilitation Project successfully addressed several threats to the reliability of a major aqueduct including rock falls, talus slopes, and potentially unstable cross slopes. On a cliff reach needed for hydropower and spillway functions, the old pipe was concrete-encased and bolted to the slope to provide reliability and driving access. To reduce mass and improve aesthetics, parts of the pipe were relocated into the slope, large voids were filled with polystyrene foam, and a box culvert alternative to the existing pipe was allowed. On steep native cross-slopes, precautions were taken to prevent local slope failures near the pipe envelope by soil nailing uphill slopes which needed to be cut to install the large 3.05 meter (120-inch) diameter replacement pipe below ground.

Twin 30-inch Ductile Iron Pipe HDD Crossings of the Historic San Marcos River

By: Kathy Pontesso, P.E.[1] and Ralph Carpenter[2], M.ASCE

Abstract: Population growth and, consequently, increased demand on the water distribution system in San Marcos, Texas, created the need for increased water flow to the southeast section of the city. The City's Master Plan defined the size and established preliminary termination points for the proposed transmission main. Although the Master Plan defined the pipe size as 30-inch (762mm), it was the responsibility of the City of San Marcos's consultant, Carter & Burgess, Inc. (Engineer), to confirm the size and to provide a final design for the water transmission main.

The project final design called for approximately 19,500 feet (5.9 km) of 30-inch (762mm) and 5,200 feet (1.6Km) of 24-inch (610mm) water line.

Mitigating the impact to historical and cultural resources was paramount to this project. A major portion of the proposed pipeline alignment followed the El Camino Real, or Kings Highway, a trail from Monterrey, Mexico, to San Marcos, to the Caddo Indian villages in East Texas. This trail was used by the Indians of yesteryear and many Spanish explorers who traveled along the San Marcos River. One site within the project corridor was the Villa of San Marcos de Neve, established in 1808. Previous archeological excavations near this site found artifacts dating back more than 11,500 years. Today, the old Camino Real is one of Hays County's most valuable historical assets.

To ensure preservation of this protected area, the Engineer specified horizontal directional drilling (HDD) for construction of the pipeline under the San Marcos River and through the area. The HDD installation portion of this project presented several unique opportunities and challenges. This paper will discuss several of these, including the world's first installation of twin parallel 30-inch (762mm) flexible restrained joint ductile iron pipelines, each approximately 1,200 LF (369m).

The project also involved the Ductile Iron Pipe Research Association (DIPRA) and Corrpro, a leading provider of corrosion control engineering services, systems, and

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equipment, to evaluate the long-term effectiveness of polyethylene encasement applied to 30-inch (762mm) pipeline installed using HDD.

Finally, this paper will provide designers and end users with information about the proper HDD application, design, and installation of flexible restrained joint ductile iron pipe.

Preserving History: In January 2004, Carter & Burgess, Inc. was selected to design a 30-inch (762mm) potable water transmission main for the City of San Marcos, Texas. This project presented a unique opportunity to make history while also preserving it. The initial route studies suggested that the most cost effective alignment crossed the San Marcos River at Old San Antonio Road a location that was once the site of Villa de San Marcos de Neve.

San Marcos de Neve, founded in 1808 as one of the last Spanish attempts at colonization in Texas, was located along the San Marcos River just four miles below the present location of the City of San Marcos in central Texas. It was intended as part of a chain of defensive settlements stretching along El Camino Real (Kings Highway), a colonial trail that was contiguous from Caddo Indian settlements in east Texas all the way to the Mexican city of Monterey. According to the *Handbook of Texas Online*, "estimates of the villa's size vary from about fifty to eighty people, including perhaps a dozen families and servants and as many as 1,700 animals -- cattle, horses, and mules."[a]


Figure 1 - San Marcos River at the location of Villa de San Marcos de Neve.

The small struggling colonial community of San Marcos de Neve was both blessed and cursed by its location near the San Marcos River. The river blessed the community by providing a source of clear, cold, artesian spring water that flowed from the underground Edwards Aquifer and produced the water that flows down the San Marcos River. The river was a curse in that it also collected storm run off that nearly wiped out the nascent community in June 1808. Then, after just a few years of flourishing, harassment by Comanche and Tonkawa indians overwhelmed the last of the settlers and the community was abandoned circa 1812. Over the years, explorers, traders, indians, and settlers who stopped to quench their thirst at this site left behind artifacts that are a reflection of the rich history of Texas and our country.

In an effort to delimit the three-dimensional boundaries of this treasured and historical site, the Engineer contracted with Texas State University at San Marcos (TSU) to perform an archeological assessment of the site. TSU had previously investigated several sites in the area of the proposed crossing so its experience would be valuable in this site assessment and preserving the history of the site.

The report from TSU dated, September 2005, outlined areas of historical significance along the proposed waterline alignment. This report also made recommendations for preservation of the historically significant deposits near the river and near the site of the original community of San Marcos de Neve. Utilizing horizontal directional drilling allowed the deposits of historically significant materials to remain in place, undisturbed, during construction.

The Plan for Making History: Given the parameters for preserving the antiquity of the site, completing the plan and profile for the HDD portion of the transmission line was rudimentary. The profile of the line indicated a maximum depth of 60 feet (18.3m). This depth was based on a curve radius of 1,800 feet (548m), which is considered very tight for the size drill rig required for this project. Ideally, HDD contractors would prefer radii that range from 50 feet (15m) per one-inch (25.4mm) of nominal diameter to 100 feet (30.5m) per inch (25.4mm) of nominal diameter. These preferred radii are a function of the limits of the drill rod and not a limitation of the flexible restrained joint ductile iron pipe. Table 1, below, provides dimensions and allowable capacities for AMERICAN Flex-Ring® joint pipe. The allowable deflections are for full length pipe. However, if tighter radii are required, Flex-Ring can be fabricated in half-lengths to effectively reduce the radii even further. Considering the dynamic stresses that drill rods are subjected to including rotating/torsion, bending, and the constant stress of either tension or compression during reaming and pipe pullback, it's understandable why HDD contractors prefer the larger radii. HDD contractors also prefer to avoid any bore that could dynamically fatigue their drilling rods.[b]

Size in.	Working Pressure* psi	Nominal Laying Length** ft.	A O.D. in.	B Socket Depth in.	F Bell O.D.+ in.	Allowable Pulling Load++ lb.	Allowable Deflection degree	Offset per 20' Length in.	Radius of Curve^ ft.	Empty Pipe Buoyancy in Water (lb/ft)^^
4	350	20	4.8	5.62	7.06	10,000	5	21	230	-5
6	350	20	6.9	5.62	9.19	20,000	5	21	230	-2
8	350	20	9.05	5.74	11.33	30,000	5	21	230	3
10	350	20	11.1	6.72	13.56	45,000	5	21	230	11
12	350	20	13.2	6.72	15.74	60,000	5	21	230	19
14	350	20	15.3	7.38	19.31	75,000	4	17	285	27
16	350	20	17.4	7.38	21.43	95,000	3 3/4	16	305	38
18	350	20	19.5	8.2	23.7	120,000	3 3/4	16	305	52
20	350	20	21.6	8.2	25.82	150,000	3 1/2	15	327	69
24	350	20	25.8	8.96	29.88	210,000	3	12	380	104
30	250	20	32	9.63	36.34	220,000	2 1/2	10	458	175
36	250	20	38.3	9.63	42.86	310,000	2	8	570	266
42	250	20	44.5	10.84	49.92	390,000	2	8	570	359
48	250	20	50.80	12.37	56.36	500,000	2	8	570	484

Table 1 - Flex-Ring Joint Dimensions and Capabilities

Prior to the preparation of bid documents, the Engineer was asked to append a bid item for an additive alternate for a second parallel crossing of the San Marcos River approximately 60 feet (18.3 m) down river from the location of the proposed treated water transmission line for the City of San Marcos. The present line crossing the river is a 24-inch (610mm) HDPE pipeline that has a restrictive inside diameter of 19.374 inch (491mm). This parallel line would be a twin 30-inch (762mm) raw water line that would be installed by HDD and serve as a redundant crossing for the Guadalupe Blanco River Authority (GBRA) that supplies raw water to the City of San Marcos water treatment plant.

In preparing specifications for the project the Engineer had to prepare the pipe specification providing options for the primary HDD crossing and the alternate one. Operational parameters differ for each of the twin 30-inch (762mm) lines. For the treated water transmission line the elevated operating pressure was a controlling factor. This line, originating at the City of San Marcos' water treatment plant and pump station has the highest pressure of the 2 lines. The hydraulic gradient at the indicated depth equated to a working head of approximately 460 feet (140.2m) or about 200 psi (1.4MPa). At this pressure the use of high density polyethylene (HDPE) pipe would be prohibitive as the maximum allowable pressure for this size of visco-elastic plastic, HDPE pipe, is 160 psi (1.1 MPa). The pressure limitation of HDPE pipe was not an issue with the parallel 24-inch GBRA raw water line.

Searching for options the engineers were aware of several very successful HDD installations of flexible restrained joint ductile iron pipe in Texas. In contacting several of the references provided by American Ductile Iron Pipe (Manufacturer), including owners, engineers, and contractors involved with the referenced projects, the Engineer was able to make the observations and comparisons shown in Table 2 above.

Ductile Iron Pipe	HDPE Pipe
Pressure capability up to 350 psi (2.4 MPa).	Pressure limited to maximum of 160 psi (1.1 MPa) with 30-inch (762mm) pipe.
Pipe wall is impermeable to hydro carbons, less potential for contamination	Permeation of the wall is highly probable in contaminated soils.
Flexible restraint joint effectively redistributes pulling and flexural tension stresses due to bending at each joint.	Flexural tension due to bending is additive to the pulling tension.
Sectional joints allow an option of either assembled-line or cartridge assembly.	Must use the assemble-line method of assembly where all joints are fused together prior to pullback.
Elastic material, strength does not decrease with time.	Visco-elastic material, because of creep, the strength of the pipe decreases over time, with no option for definitive life extension
In aggressive soils PE encasement and the addition of joint bonding affords the potential for the application of life extending cathodic protection, if necessary.	Inert material, unaffected by aggressive soils.

Table 2 - Summary of Observations and Comparison - Ductile Iron Pipe and HDPE Pipe

Specifications for the City of San Marcos treated water transmission line required that only flexible restrained joint ductile iron pipe be utilized due to the pressure requirements. The GBRA raw water main was originally specified as HDPE pipe to mirror its existing 24-inch (610mm) raw water line.

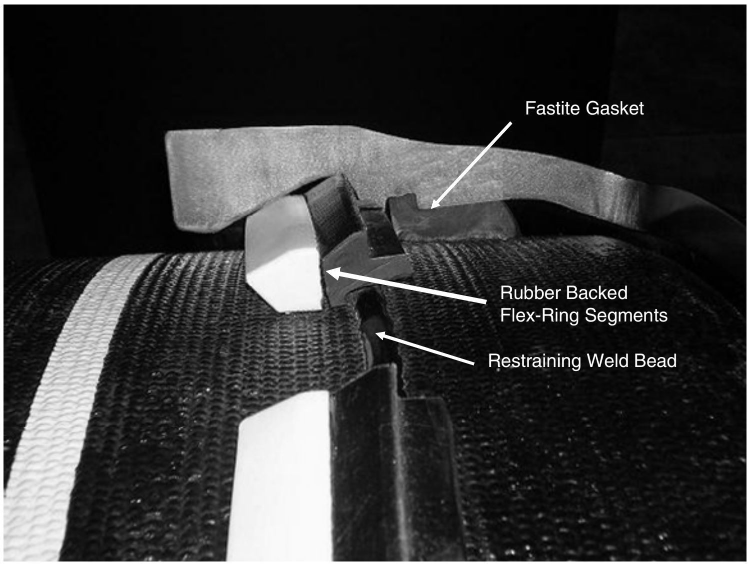

Figure 2 - AMERICAN Flex-Ring® Joint Ductile Iron Pipe

Preparation of the specifications was aided by use of a guidelines document, available from the Manufacturer that highlighted some HDD caveats. One of these caveats identified the importance of having unobstructed flow over and around the bell during pipe pullback. The Manufacturer suggests that joints used for directional drilling should be boltless, flexible, restrained and should only be AMERICAN Flex-Ring® or approved equal. Any joint with bulky glands or flanges that may prevent the smooth flow of the drilling fluid/soil slurry over the joint should not be used. The Flex-Ring joint, as shown in Figure 2, illustrates the desired smooth transition over the joint.

Corrpro Companies, Inc., the one of the premier providers of corrosion engineering services, was brought in by the Engineer's design team to consult on corrosion control for this project at the proposal preparation stage. In 2004, after significant research, Corrpro and the Ductile Iron Pipe Research Association (DIPRA) jointly developed the Design Decision Model™ (DDM™) a risk-based matrix that incorporates the likelihood of corrosion and the consequences of a possible corrosion-related failure.[c] This project is an excellent example of how to implement the DDM™. Normally, if the project was to be installed using open-cut construction, with minimum amount of cover over the top of pipe, the soils would dictate that the ductile iron pipe be encased with loose polyethylene encasement. However, since this HDD crossing is 60 feet (18.3m) deep, in the middle of the San Marcos River, and in a historically sensitive area in

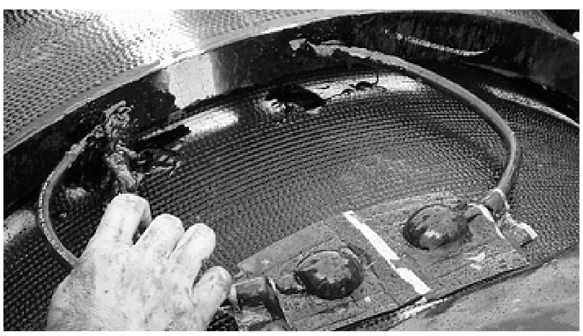

Figure 3 - Flex-Ring Joint with bonding

Hays County, Texas, the line fell into a high-risk category. As a result of analyzing the risk based on consequence and likelihood of a corrosion related issue, the twin 30 inch (762mm) Flex-Ring joint ductile iron pipe crossings would require PE encasement, joint bonding for electrical continuity (see Figure 3) with electrical test stations, and life extending cathodic protection using sacrificial anodes.

It is important to note that thanks to the cooperation of the City of San Marcos, GBRA, Corrpro, DIPRA, and the Engineer, both of these HDD crossings will be a part of a long-term study on the effectiveness of PE encasement on HDD installed ductile iron pipelines. Monitoring of the pipeline will be performed by DIPRA field engineers and coordinated with the City of San Marcos.

The project, City of San Marcos El Camino Real Water Transmission Main, was bid on August 11, 2005. The successful bidder was Key Enterprises, Inc. (Key), with a bid of $3,523,454.25 for the project that had an engineer's estimate of $4.5 million. The primary difference in the bid price and the engineer's estimate was due to a decrease in the installed cost of ductile iron pipe as compared to recent projects of similar size and scope. The additive alternate for the GBRA HDD crossing was bid at $403,157 and was awarded with the base bid items. Key Enterprises chose to use Trans American Underground – HDD (TAUG), Flower Mound, Texas, for its HDD subcontractor.

The HDD installation involved a cooperative effort between Key and its subcontractor TAUG. TAUG was responsible for excavation of the borepath and pipe pullback, while Key was responsible for the purchase of AMERICAN 30-inch (762mm) Flex-Ring®, flexible restrained joint pipe, all necessary excavations, and pipe assembly.

With pipe of this size, pipe assembly can be by either the assembled line or cartridge assembly method. The assembled line method requires the pipe to be pre-assembled and pulled back through the borepath as a single unit. The cartridge method, by comparison, is when individual sections of flexible restrained joint ductile iron pipe are assembled and then pulled into the borepath a distance equivalent to the length of a single pipe section. This process would be repeated for each pipe section until the entire line was pulled through the bore path to the exit point. Some contractors consider the time required for joint assembly as an added risk. Experience has shown that if the borepath has been properly reamed and the appropriate drilling fluid has been used to stabilize the in situ soils throughout the borepath that the risk associated with the cartridge installation method is not significantly greater than it is for disassembly of the sections of drill rod. Assembly cycles for the Flex-Ring joint can range from just a few minutes up to 8 minutes depending on any joint requirements i.e.: bonding jumpers, polyethylene encasement, etc. Assembly utilizing the cartridge method was selected due to the restricted right of way and the requirement to maintain unobstructed, continuous traffic flow on Old San Antonio Hwy.

TAUG mobilized to a site located approximately 350 feet (107m) south of the San Marcos River along Old San Antonio Road on March 20, 2006. This location was ideally suited for this project as the property owner and TAUG negotiated an agreement to dispose of the spent drilling slurry by land application on a site adjacent to the drilling location where the Tulsa Rig Iron mud recovery system was set up to recycle the bentonite slurry. The composition of this drilling slurry is primarily naturally occurring material that is blended with the water-bentonite drilling fluid, which has the National Sanitation Foundation (NSF) certification per NSF-61.

HDD Practices	Detail
Assembly Method assessment	Define and describe the Cartridge and assembled line methods used with the installation of ductile iron pipe.
Buoyancy	Compensating for positive buoyancy by adding internal weight by various methods.
Application of PE (When Required)	Apply PE encasement per AWWA C105 Method B for sub-aqueous installation and as modified by the manufacturer.
Joint Bonding (When Required)	Procedure for applying copper bonding jumpers: 1. Drill and tap for 2 ea (Min) 1/4in bolt in vertical face of bell, mark location 2. Grind and CAD weld copper bonding jumper to spigot end. 3. Apply insulating caps to CAD welds.
Borepath Transition	Cover methods of transitioning from horizontal to entrance ramp to avoid over-deflection of joints.
Pipe Handling	Handling PE encased pipe using padded forks and mast; and also non-abrasive lifting slings.
Pre-Assembly and Assembling Flexible Restrained Joints	Pre-assembling components of the flexible restrained joint including the sealing gasket into the bell; and rubber-backed restraining segments into the bell. Assembling of the joint, proper engagement of restraining segments, visual inspection for dislodged sealing gasket.
Joint Finishing 1. Joint Bonding 2. PE Encasement	Proper alignment of bonding straps and tapped holes and application of insulating mastic. Wrapping of PE encasement per the pipe manufacturers suggested guidelines for installation using HDD.

Table 3 - Pre-Installation Crew Training

As a service to its customers, the Manufacturer provides the services of a product engineer or other experienced personnel to instruct installing contractors on the most cost effective methods for installing ductile iron pipe using HDD for all HDD pipeline installations in this size range or larger. Key had significant experience history with ductile iron pipe, and its crews were very familiar with the assembly of their patented flexible restrained joint. However, there were some additional practices that are common for pipe being installed using HDD that they were not aware of. Therefore, on April 4, 2006, a product engineer from the manufacturer met with Key's project superintendent and installation crews to discuss and demonstrate these practices. See Table 3 above for more specific details.

The calculated buoyancy force for the 30-inch (762mm) Flex-Ring in drilling fluid was approximately 175 pounds (65Kg) per linear foot positive buoyancy. The most ideal situation would be to have the pipe neutrally buoyant during pipe pullback so there would be little or no normal resistant frictional forces once the pipe entered the soil-slurry medium. TAUG and Key decided not to add any additional internal weight, which would have been approximately equivalent to adding a 1200 foot (366m) section of 18-inch (457mm) HDPE to the interior of the Flex-Ring joint pipe

The first of the twin lines to be installed was the treated water line for the City of San Marcos. This section is the longest of the twins at 1200 feet (366m).


Figure 3 - Cartridge Assembly On Pipe Ramp

In brief, HDD is a three (3) step pipeline installation methodology that includes: 1) pilot bore, 2) reaming, and 3) pipe pullback. Using American Augers' DD330 drilling equipment, which is capable of pulling or pushing with a force of 330,000 pounds (123 metric tons), TAUG directionally bored a pathway from the south side of the San Marcos River to its north side using a 9-inch (229mm) jettable steering head. This head was connected to a 5-inch (127mm) outside diameter, 30-foot (9m) long steel drill pipe. This drill pipe was rotated to blend the excavated silty-clay soil with the drilling fluid and pushed to advance the drill string in a straight line. To change direction, the steering head was positioned in a specific "clock" position and advanced without rotation. As stated before, The Engineer's original design had the minimum radius of the bore path at 1800 feet (549m), however, in discussion with TAUG, The Engineer compromised on an installed radius and allowed radii between 2000 and 2500 feet (610m – 762m). The in situ soils proved to be an excellent soil medium for directional drilling.

Drilling fluid has multiple purposes. First, it serves as a hydraulic power source and coolant for drilling heads (called "mud motors"), particularly in rock. Next, by using the appropriate mixture of water, bentonite, and polymer, the drilling fluid blends with the soil, holds it in suspension and moves it to the exit and entrance pit for recirculation. The drilling fluid in some soils produces a soil cake that stabilizes the inside diameter of the borepath. Finally, the drilling fluid acts as a lubricant for the finished pipe during pipe pullback.


Figure 4 - Flex-Ring Pipe Ready for Pullback

After completing the pilot bore without incident, TAUG then ran the first of three reamers through the borepath. The purpose of this reaming procedure is to enlarge the borepath to a final inside diameter of approximately 48 inches (1200mm). The first pass through the pilot bore path enlarged the pathway to 24 inches (610mm). Drilling continued with subsequent passes of the 36-inch (914mm) then the 48-inch (1200mm) reamers. Aware of the importance of a good borepath for the pending pipe pullback, TAUG operators "swabbed" the line by pushing a barrel reamer or packer back through the borepath.

As with any construction project, the Critical Path Method (CPM) requires that many operations occur simultaneously to meet the project's completion schedule. Such was the case with the twin HDD installations on this project. As TAUG continued drilling and swabbing the borepath, Key's crew was busy preparing the pipe for assembly by applying the PE encasement, and pre-installing both the Fastite® gasket and the rubber backed Flex-Ring segments (see Figure 2). Application of the PE encasement requires the contractor to center the PE tubes on the pipe section, remove any slack by folding the PE inward, then secure the wrap tight to the pipe barrel with either circumferential or helical wraps of duct tape. On Tuesday, April 11, 2006 one of the pipe laying crews began preparation of a ramp down to the borepath entrance. The crew prepared for the cartridge assembly method – that is, assembling one joint at a time. Key's crew used a series of five pipe rollers, approximately 20 feet (6m) apart, to assist in pipe assembly. These rollers allowed the crew to more efficiently complete the connection of the bonding jumpers, and complete the overlap and taping of the polyethylene wrap according to the procedure outlined in Suggested General Guidelines – Horizontal Directional Drilling With Ductile Iron Pipe[d]. The additional rollers also allowed Key to take advantage of any downtime on the American Augers DD-330 drill rig and assemble multiple joints. This occurred several times during the pullback when two (2) or more sections of drill rod would lock up and require extra effort to disconnect.

Making History: On Wednesday afternoon, April 12, 2006, Key and TAUG prepared for the final step in the HDD process, pipe pullback. The 5-inch (127mm) drill rod with the fly cutter and barrel reamer still connected was pushed up the entrance ramp. Then the swivel, which isolates the rotation of the drill rod from the new Flex-Ring joint pipe, was attached and in-turn connected to the Flex-Ring pulling head that had already been assembled onto the first section of Flex-Ring joint pipe. Several more pipe sections were assembled before the reamers disappeared into the bore path on the start of the 1200 foot (366m) trek under the relics of history being preserved thanks to this HDD installation. It was approximately 4-1/2 hours after the start of the pullback that the 60th joint was assembled and pulled back toward the entrance to the borepath completing the HDD portion of the City of San Marcos treated water transmission line. TAUG indicated that the in situ soil conditions were excellent and that at no time did the pulling loads exceed 54,000 pounds (20.2metric tons). Using Equation 1 the calculated coefficient of friction was 0.26 indicating that the PE encased ductile iron pipe offered very little resistance due to frictional drag.

Equation 1 – Calculating the Empirical Value for μ

$F_P = \mu (W_B)(L)$ solving for μ

$$\mu = \frac{F_P}{(W_B)(L)} \qquad (1)$$

Where:
F_P = pulling force, lbs.
μ = coefficient of friction between pipe and slurry between pipe and ground (typically 0.40 to 0.60)
W_B = net unit downward (or upward) normal force on pipe, lb/ft
L = pull length, ft

As stated earlier, the City of San Marcos HDD pipe was specified as restrained joint ductile iron pipe, and the GBRA raw water transmission line was specified as HDPE SDR 11 to mirror their existing 24-inch line installed by HDD in 2001. On August 29, 2005, Hurricane Katrina made landfall just south of New Orleans, causing extensive damage to a large portion of the Gulf Coast. A substantial portion of this country's HDPE and PVC resin is produced in this region, and virtually all of the production was halted for a period that extended well into 2006. This lack of supply resulted in severe resin shortages for the production of pipe and commitments for deliveries and pricing were non-existent. It was during this period when GBRA requested that its raw water line be upsized from 24-inch (610mm) to 30-inch (762mm). Considering the size change and the uncertainty of delivery, project engineers made the recommendation and proceeded to modify the design for the GBRA raw water line to 30-inch (762mm) flexible restrained joint ductile iron pipe.

On April 12, 2006, installation began on the second of the two (2) parallel HDD lines that were being installed to provide a larger, redundant crossing of the San Marcos River for GBRA's raw water transmission delivery system. This CPM was implemented much like the first HDD crossing with the site, equipment, and crews from Key and TAUG ready for pipe pullback on the morning of May 15, 2006. Approximately 5 hours later, the 58th section of Flex-Ring joint pipe was pulled down the entrance ramp completing the 1160-foot (354m) to TAUG pullback loads were lower for this second HDD lines. Loads never exceeded 43,000 pounds (16 metric tons). Using Equation 1 the value for the effective coefficient of friction (μ) was 0.21, again indicating that the PE encased flexible restrained joint ductile iron pipe offered very little frictional drag.

Observations and Conclusions: The primary purpose of this line was to deliver raw and treated water to the modern day inhabitants of the City of San Marcos without disturbing the areas where the colonial inhabitants of San Marcos de Neve and those who traveled the El Camino Real left their indelible marks of existence. In the process of preserving this history, the successful HDD installation of the twin 30-inch (762mm) lines made indelible marks on the history of ductile iron pipe. For now, this installation has made its mark as the world's longest (1200 foot or 366 m) HDD installation of 30-inch (762mm) ductile iron pipe.

Ductile iron pipe, with a family tree dating back to the 15th century, is focused on changing and adapting to the demands of present and future generations. Trenchless installation is a perfect example of the industry's commitment to change while always preserving the history of past generations.

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[a] *Handbook of Texas Online*, s.v. ",," http://www.tsha.utexas.edu/handbook/online/articles/SS/hvs21.html (accessed December 2006).

[b] Ma, et al, 2005, "Fatigue of Drill Pipes Used in Mini-Horizontal Directional Drilling," Proceedings of the ASCE Pipeline Division Conference, Houston, TX.

[c] Kroon and Lindemuth, 2004, "Corrosion Protection of Ductile Iron Pipe," NACE 2004

[d] Carpenter and Conner, 2005, "Suggested General Guidelines – Horizontal Directional Drilling (HDD) Installations of Ductile Iron Pipes", Birmingham, AL.

Pipe Jacking in Difficult Urban Waterfront Conditions

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Abstract

Installation of combined sewer overflow (CSO) consolidation conduits in difficult waterfront conditions would have been no easy task as originally designed and bid, using cut-and-cover methods, yet an aggressive Value Engineering (VE) proposal was accepted and the installation accomplished using pipe jacking methods. Five-hundred-and-ten (510) feet of conventional pipe jacking was successfully accomplished to install 54-inch and 30-inch consolidation conduits as part of the Narragansett Bay Commission's (NBC) CSO Control Facilities Program.

This paper will present site conditions, a comparison of the original cut-and-cover design and the VE pipe jacking proposal, provide construction details and challenges. The conduits were installed in a narrow strip of land between historic waterfront structures and the Providence River with a portion of the alignment beneath high-tension towers and transmission lines. The ground conditions consisted of urban fill, a soft organic silt deposit, and abandoned power tunnels, slips and wharfs. Bid documents directed an open cut with an Owner-designed secant pile wall support of excavation and installation of reinforced concrete pipe (RCP) for the conduits, and the Contractor's proposed VE largely eliminated open cut and included pipe jacking with sheeted pits. The VE proposal was negotiated and the Contractor successfully completed the pipe jacking using an open-face shield, ground improvement using chemical grouting, Permalok steel casing and high density polyethylene (HDPE) pipe. Several large obstructions, including concrete-filled vaults, abandoned tunnels, timber piles, and steel sheet piling were encountered and excavated from within the pipe jacking shield. Challenges and risks associated with open-cut excavation in a difficult waterfront environment were significantly reduced by implementing the pipe jacking VE change, but were not entirely eliminated.

Project Overview

The NBC is a quasi-public regional sewer authority that serves 10 communities in the greater Providence, Rhode Island, metropolitan area. From 1992 to 1999, the NBC developed a comprehensive facilities plan to abate CSO pollution in the Upper Narragansett Bay. In 1999, a final plan was accepted. The final plan included two deep tunnels, five near-surface CSO interceptors, 12 sewer separation projects, a wetland facility, and a wastewater treatment facility upgrade. The program was to be constructed over 20 years in three sequential phases. In 1992, the NBC retained Louis Berger Group as Program Manager (PM) for the CSO Abatement Program. The Phase 1 PM team included Jacobs Civil Inc. as the primary designer, assisted by CH2M Hill, Inc. Haley & Aldrich, Inc., provided geotechnical design, and Gilbane/Jacobs Associates construction management services.

Phase 1 of the program, included a 3-mile-long, 26-foot-diameter, 230-foot-deep storage tunnel, adits, pump station, and seven near-surface diversion and relief facilities. Construction began in 2001, with facilities to be operational by 2008. Each of the seven near-surface facilities was constructed concurrently with the tunnel contract and included overflow diversion or interceptor relief structures, consolidation conduits, and drop shafts to convey combined sewer flows to the storage tunnel.

This paper focuses on the trenchless technology used in connection with construction of one of the near surface facilities. Pipe jacking was used in lieu of the designed open-cut construction of two combined sewer consolidation conduits, designated Outfalls (OF) 006 and 007. The configuration of this facility is shown on Figures 1 and 2. The facility comprises two diversion structures and consolidation conduits conveying outfall overflows to a single gate and screen structure, approach channel, and drop shaft.

Site and Subsurface Conditions

Site and ground characterization work consisted of desk-top studies and geotechnical investigations to provide information that was used in both design and construction. Desk-top studies revealed that the project site had been used for many purposes over the years. In the 1800s, a wharf extended west from the waterfront. It was subsequently filled in. A portion of the alignment was also used as an electric generating facility and coal gasification plant. Active and abandoned underground infrastructure remain at the site.

Geotechnical investigations included several test borings along the alignment. In general, encountered ground conditions indicated that the alignment was underlain by 15 to 20 feet of fill, 10 to 30 feet of organic deposits, followed by estuarine deposits, glaciofluvial deposits, and glaciolacustrine deposits. Conduit inverts were anticipated to be primarily in the organic deposits and 8 to 11 feet below the groundwater table.

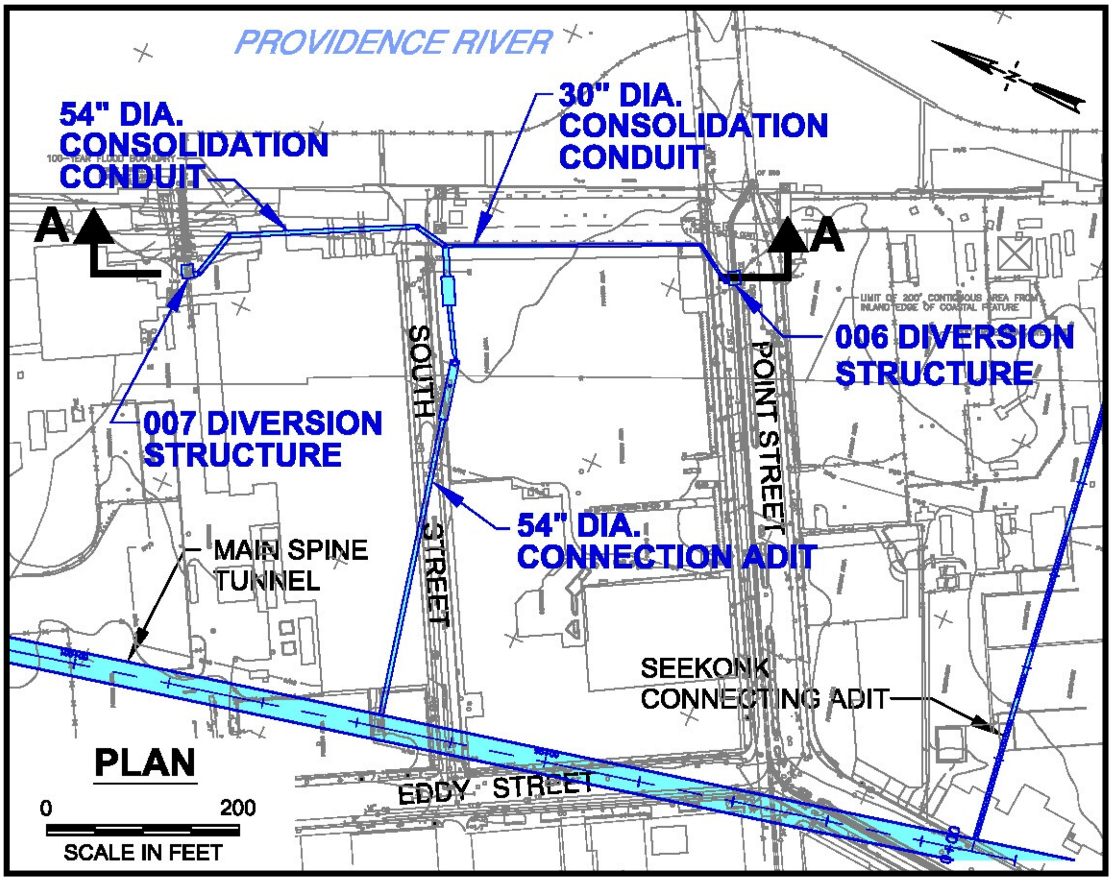

Figure 1. General Plan.

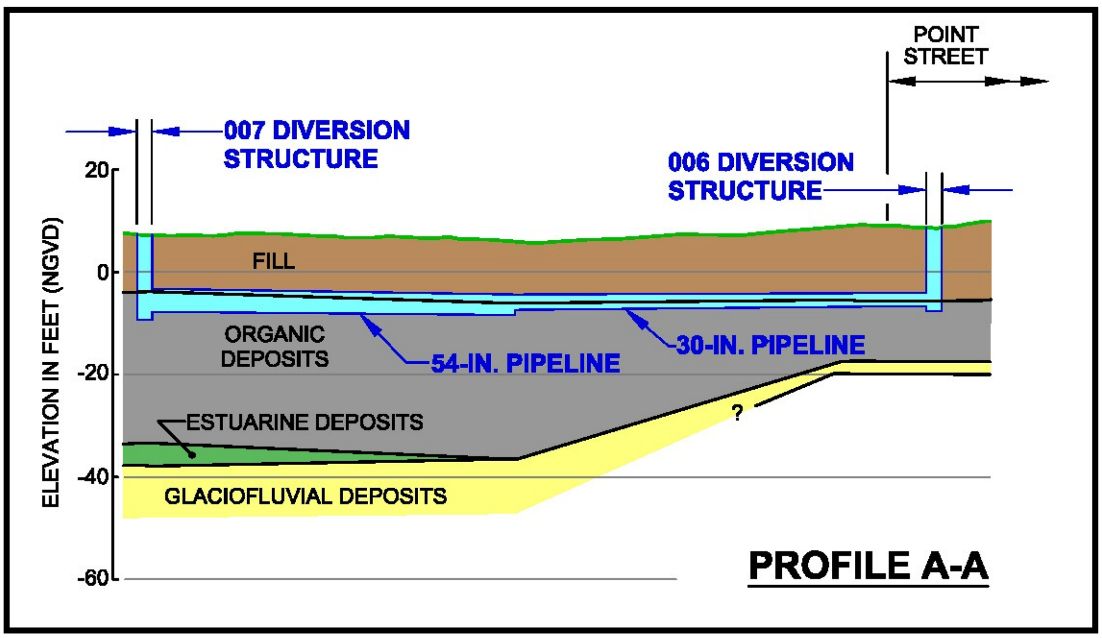

Figure 2. Subsurface Profile.

The soil strata encountered within the pipe zone of influence are described below.

Fill: The fill typically consisted of loose to dense, brown to black coarse to fine sand with varying amounts of silt and gravel. The fill was also noted to contain building debris and other deleterious materials in varying amounts. Such materials include cinders, brick, ash, coal, slag, concrete, and wood. As indicated above, live and abandoned infrastructure was expected to be within the fill.

Organic Deposits: The organic deposits varied in description. Typically, the unit can be described as a very soft to stiff organic silt with trace to little clay, fine sand, and shell fragments. The organic content of this deposit ranged widely from 3.5 to 31.1 percent.

Environmental investigations indicated that the soil and groundwater were significantly contaminated by the previous use of the site for coal gasification and power generation. Very high levels of mercury and petroleum hydrocarbons were found.

Arriving at a Value-Engineering Proposal for the Pipejacking Method

Many constraints affected the selection of the drop shaft site and the consolidation conduit routes. Ultimately, these constraints made it necessary to abandon the original conduit design proposal and adopt a value-engineering proposal based on the pipejacking method.

The proximity of the site to the river, the fill and silty organic soils, and the maze of decommissioned structures and tunnels in the project area were all major obstacles. In addition, the proposed corridor was located below critical power supply lines and transmission towers, adjacent to the power station that was to be renovated into the museum, adjacent to an operating pub, and in conflict with future underground power-lines. Compounding all of these issues was the significant contamination on the site, which made all activity subject to strict environmental regulation.

After a protracted negotiation between NBC and the site owner, investigation was allowed to proceed, contingent upon a constructability analysis. As the project management team proceeded with its analysis, it became clear that assuring the interim and long-term stability of adjacent structures was the primary determining factor; the selection of a pipe alignment and pipe support began to be highly influenced by construction methods.

Throughout the constructability analysis, there was much debate over the relative merits of trenchless versus open-cut methods. Because work would be done under power lines and between the tower's legs, a method that minimized or eliminated the use of surface equipment was highly desirable. However, the extreme variability of the ground and the presence of readily recharging groundwater made it difficult to select a trenchless method. Although the open-cut alternative would have to deal with the same conditions, it was recognized that the pipe and excavation support could be more readily aligned to avoid known structures.

The team determined that it would be feasible to construct a secant pile wall using low-profile equipment for the open-cut excavation. Concurrently, obstacles could be demolished and removed from the surface. The secant pile wall would provide foundation support when tied into a concrete slab that ran under the RCP. Contract documents prescribed this design for the OF 007 conduit. For the OF 006 conduit, the alignment was routed slightly to the west of the power lines, H-pile foundations were specified, and the selection of soil support method left to the Contractor. A substantial array of geotechnical instrumentation and a stringent monitoring protocol were also selected to demonstrate to the electric utility that existing structures would be protected from disturbance.

The electric utility had strongly urged a complete prohibition on surface equipment, but the PM team wanted to avoid restricting options to trenchless-only methods. Terms for construction and permanent easements were successfully negotiated with the electric utility and the design was finalized in 2002. Choosing the secant pile option meant that the contract documents contained an appropriately conservative scope of work for bid. When “alternate” methods of construction were raised during bidding, the bidders were informed that alternatives would be conditional to approval by both the NBC and the electric utility. A successful alternate would need to equal the performance of the secant wall in that it would have to: provide greater protection against movement of the adjacent existing structures; provide sufficient water cutoff to limit water drawdown outside the excavation; negotiate known and unknown underground obstructions; maintain safe overhead clearances; and provide permanent structural support to the conduits.

Immediately following the notice to proceed on 2 January 2003, the general contractor, Barletta Heavy Division (BHD), raised a proposal to jack a steel pipe with an open-face shield and line it with a grouted HDPE carrier pipe. BHD’s engineering consultant, GZA GeoEnvironmental, Inc., gathered additional data from borings, test pits, and archival research, and tested the soil’s permeability, strength, and compressibility. It demonstrated that short- and long-term settlement of an unsupported conduit would be within the contract tolerances for finished pipe and that the hydraulic function would be achieved using HDPE. Subsequent review by the project management team focused on satisfactorily addressing the above-mentioned performance requirements and additional issues specific to the proposed method. During review, concerns regarding the effectiveness of well point dewatering led to the adoption of chemical grouting as the primary method to control groundwater at the tunneling face. Borros points were also added to monitor ground movement above the pipe.

Having recognized the advantages of less risk to its facilities, the electric utility did not object to the change of methods. Thereafter, the NBC and BHD negotiated the contractual terms for the change for the OF 007 54-inch conduit as well as modified specifications and submittal requirements. The change language contained a waiver of the DSC clause, a waiver from claims pursuant to the change of method, and acknowledgement that unit prices for obstruction removal (which had been intended for surface construction) would not be reimbursed for the jacking option.

BHD also favored jacking for the OF 006 conduit installation and similar contract conditions were agreed upon for the OF 006 30-inch conduit. Additionally, the OF 006 diversion structure was shifted to the east, reducing the length of conduit and resulting in a credit. It was recognized that this shift also moved the work closer to the Goff's structure, a single-story masonry building built in the 19^{th} century. The change order language therefore also stated that should damage be caused by the change of methods, the Contractor would cover the deductible of the Owner-controlled insurance coverage.

Construction

Installation of the consolidation conduits was completed by jacking Permalok steel casing using an open-face shield and subsequently installing HDPE carrier pipe. The 54-inch conduit was 250 feet long and the 30-inch conduit was 260 feet long. The elevation of existing outfalls 006 and 007 controlled the elevation of the consolidation conduits – which were approximately 22 feet deep - situating them partly in the organic silt deposit and partly in the fill. During the installation, the ground conditions proved more difficult than anticipated, and additional ground improvement and significantly more effort were required to complete the installation. Below, the selection of the methods, method details, and production rates and adjustments are discussed.

Selection of Methods

BHD contracted with M&P Pipe Jacking Corporation (M&P) of Newington, Connecticut, to complete the pipe jacking. M&P and its proposed personnel were selected because they had demonstrated successful experience jacking through similar difficult ground conditions.

Anticipated man-made structures included: abandoned vaults, channels, and tunnels connected to the adjacent abandoned power station; granite blocks and timber pilings associated with wharfs and slips buried during land expansion into the river; and typical "*urban fill.*" An open-face shield and hand excavation with full access to the face was selected, providing the ability to remove large obstacles.

The necessity to use an open-face shield, below the groundwater table and immediately adjacent to the Providence River required groundwater control for the fill material that overlay the organic silt. The shield was equipped with closable bulkhead doors, but a more positive means to control groundwater inflow during excavation was required and provided by pre-grouting the alignment using a sodium silicate-based chemical grout.

Methods Employed

Ground Improvement

BHD contracted with Hayward Baker (HB) to design and carry out the ground improvement program. To achieve a stable face and control groundwater inflows into the open-face shield, a *ground improvement envelope* with an unconfined compressive strength of a minimum of 70 psi was designed to extend from the top of the organic silt to 4 feet above the crown and laterally a minimum of 2 feet beyond the jacked casing perimeter.

HB selected a sodium silicate-based chemical grout to accomplish the ground improvement. The fill material consisted mostly of fine to medium silty sand with less than 30 percent fines. Installation of the grout pipes, consisting of 1.5-inch *sleeve port pipe*, was carried out from the surface since the anticipated numerous obstructions would be easier handled with the surface drill rig than within the shield drilling horizontally. Grout holes to install the sleeve port pipes were drilled using a hydraulic track drill tooled to be able to drill through reinforced concrete obstructions. Vertical sleeve port pipes were installed in rows of three, perpendicular to the alignment, spaced 4 feet apart with the middle pipe centered over the alignment. Figure 3 shows alignment of 54-inch consolidation conduit and sleeve port pipe pattern. Grout pipes were installed 2 feet into the organic deposit.

Two passes were used to complete the grouting, a primary and a secondary. Primary and secondary holes were assigned such that adjacent holes in both row and column would alternate as primary and secondary. A grout cutoff volume was established based on theoretical tributary volumes with adjustments for hole spacing deviations. For each grout pipe, grouting was performed from the bottom up with a double packer to grout through individual ports spaced at 15-inch centers.

Jacking and Receiving Pits

The two consolidation conduits were installed from a common jacking pit where a permanent manhole was subsequently installed to consolidate the two flows and convey it a short distance to the screening, vortex and drop structures (refer to Figure 1). The jacking pit was constructed using hot formed interlocked sheeting. No special seals were used for jacked pipe sheeting penetrations since a majority of the groundwater was controlled by the chemically grouted soil mass. The receiving pits were constructed at each of the diversion structure locations and were integrated with the sheeted excavation support system that was installed to facilitate construction of the diversion structures. The jacking pit was approximately 33 feet long in the direction of jacking and approximately 20 feet wide. Reaction blocks were constructed of 12-inch by-12-inch timbers on the equipment side and concrete filled between the timbers and inside face of the steel sheeting. Receiving pit dimensions were controlled by the diversion structures and were much larger than necessary for receiving the jacking shield and pipe.

Figure 3. Sleeve port pipe for chemical grout along 54-inch consolidation conduit alignment.

Jacking Equipment and Casing (Dimensions within this subsection are for the 54-inch consolidation conduit installation. Dimensions for the 30-inch consolidation conduit installation are summarized at the end of this subsection.)
A cutting and steering shield was manufactured from 1-inch thick 73-1/2 inch OD steel casing fitted at the rear with a casing adaptor and external ring to receive the jacking casing and with 1-1/2-inch-thick, 10-inch-long steel cutting edge at the front. The shield was articulated and housed six 50-ton steering jacks to assist in redirecting the shield head. The shield was delivered with two 20-inch-wide breasting tables and a bulkhead equipped with four hydraulically operated doors to assist in controlling ground and groundwater.

The jacking casing was the same diameter as the shield's main section, 73-1/2-inch OD and manufactured by Permalok, "The Interlocking Pipe Joining System." Casing joints are saw-toothed, flush (same inside and outside diameters as the rest of the pipe) male and female ends. Casing lengths were 15 feet, with pairs of 2-inch grout ports spaced every 4 feet along the casing and located at the 2 and 10 o'clock positions. Bentonite slurry was pumped through the ports for lubricating the casing during jacking and a cement grout was pumped through the ports at completion of jacking to fill any voids.

The shield was operated by two laborers who hand-excavated the soil as it entered the shield during jacking and guided it to a short conveyor. The conveyor dumped the muck into ½-cubic-yard cart that was winched through the jacked pipe to and from the jacking pit. Mechanical spades were used to excavate the chemically grouted fill and organic silt. Concrete splitters and torches were used to excavate the numerous obstructions encountered during jacking. Over-excavation was unavoidable when dealing with obstructions. Wood blocking was used to fill voids caused by over excavation.

In the jacking pit, six 200-ton jacks provided the thrust to jack the 73-1/2-inch OD steel casing and shield through the ground. A transit, independently secured between the jacks, was used to measure line and grade.

For the 30-inch consolidation conduit installation, a 48-inch OD Permalok pipe was jacked with a 48-inch OD cutting and steering shield similar to that used for the 54-inch consolidation conduit installation.

Carrier Pipe
The carrier pipe consisted of HDPE pipe that was installed to grade within the jacked casing and secured with blocking. The ends of the pipe were capped to allow filling of the carrier pipe with water. Once filled with water, the annulus between the steel casing and the HDPE pipe was backfilled with a cement grout.

The HDPE pipe was manufactured by ISCO Pipe with flush *Snap-Tite* male-female joints. The 54-inch consolidation conduit carrier pipe was a 54-inch OD DR 32.5 (50.7-inch ID) and the 30-inch consolidation conduit carrier pipe was a 32-inch OD DR 32.5 (30.0-inch ID). While the pipe for the 54-inch consolidation conduit has a smaller inside diameter than the name of the conduit indicates -- 50.7 inch instead of 54 inch -- analyses during the VE proposal phase demonstrated the HDPE with smaller diameter conveyed similar flow as the larger-diameter RCP it replaced.

<u>Performance, Production Rates and Adjustments</u>
Pipe jacking for the two conduits was difficult. There were many obstructions that hindered progress, including timber piles, abandoned steel sheeting, and a 15-foot-long reinforced concrete vault backfilled with concrete. There were several abandoned structures encountered that were not shown on the plans and many more timber piles than expected and heavily influenced the average production rates. One-hundred-sixteen (116) timber piles were encountered during the 54-inch consolidation conduit drive alone. For the 54-inch consolidation conduit alignment, at least 1 hour per shift was lost to obstruction removal and, as the data indicate, one-third of the shifts was entirely lost to obstruction removal. For the 30-inch, the most significant delays occurred due to groundwater inflows that required additional grouting from the surface. Table 1 summarizes the overall production rates for each drive. The 54-inch

conduit was jacked using a combination of single and double 8-hour shifts, and the 30-inch conduit using only single 8-hour shifts. Figures 4 and 5 are photographs of workers removing obstructions from within the 54-inch heading.

Table 1. Jacking Production Rate Summary

	54-inch, 250 feet	30-inch, 260 feet
Started Jacking	July 28, 2003	November 24, 2003
Completed Jacking	October 23, 2003	March 11, 2004
Number of Shifts	97 (Avg. 2.5 feet/shift)	73 (Avg. 3.6 feet/shift)
No. Shifts without any advance	30	24 (21 due to additional grouting)
No. Shifts with 2 feet or less advance	56 (Avg. 0.5 foot/shift)	34 (Avg. 0.4 foot/shift)
No. Shifts with 10 feet or greater advance	1 (11 feet/shift)	6 (Avg. 10.5 feet/shift)

Figure 4. 54-inch shield. On left side, reinforced concrete and concrete-filled sheet pile of abandoned vault. On right side, chemically grouted clean fill.

Figure 5. Chemically grouted soil is excavated by hand, and timber piles cut with hydraulic chainsaws and removed. Obstruction removal required some minor excavation ahead of shield.

There were several occurrences that had a significantly higher influence on the overall production rates. These included groundwater inflows discussed below and an abandoned vault constructed of reinforced concrete walls and backfilled with structural concrete. This vault was located along the alignment of the 54-inch consolidation conduit for approximately 15 feet at full face. The Pipe Jacking Timeline shown on Figure 6 demarcates these occurrences and their influence on production.

Methods to remove obstructions included chainsaws, acetylene torches, jack hammers, rock splitters, Brokk 150C hoe ram attachment (limited success due to space constraints), and Cardox high-pressure gas cartridges.

Conclusions

This project demonstrates the flexibility offered by pipe jacking with an-open-face shield to install infrastructure in difficult and variable ground conditions and in areas with challenging surface constraints. The open-face shield system allowed flexibility in performing additional ground improvement from the heading. A grouting program carried out in advance and from the surface proved worthwhile, considering the debris and structures encountered.

At the onset, the prime contractor, owner, and program manager took a team approach to the negotiation of the terms of the VE agreement and the completion of the work. In accordance with the terms of VE agreement, all costs incurred for removal of obstructions were the responsibility of the contractor and the owner took no credit for the proposed alternative methods. The clear delineation of responsibility included in the VE agreement led to a successful project for both the contractor and owner.

A significant component of the success was selection of specialty subcontractors with the right experience and tools to complete the work without impact to the abutting facilities. Very little, if any, movement was measured with the geotechnical instrumentation.

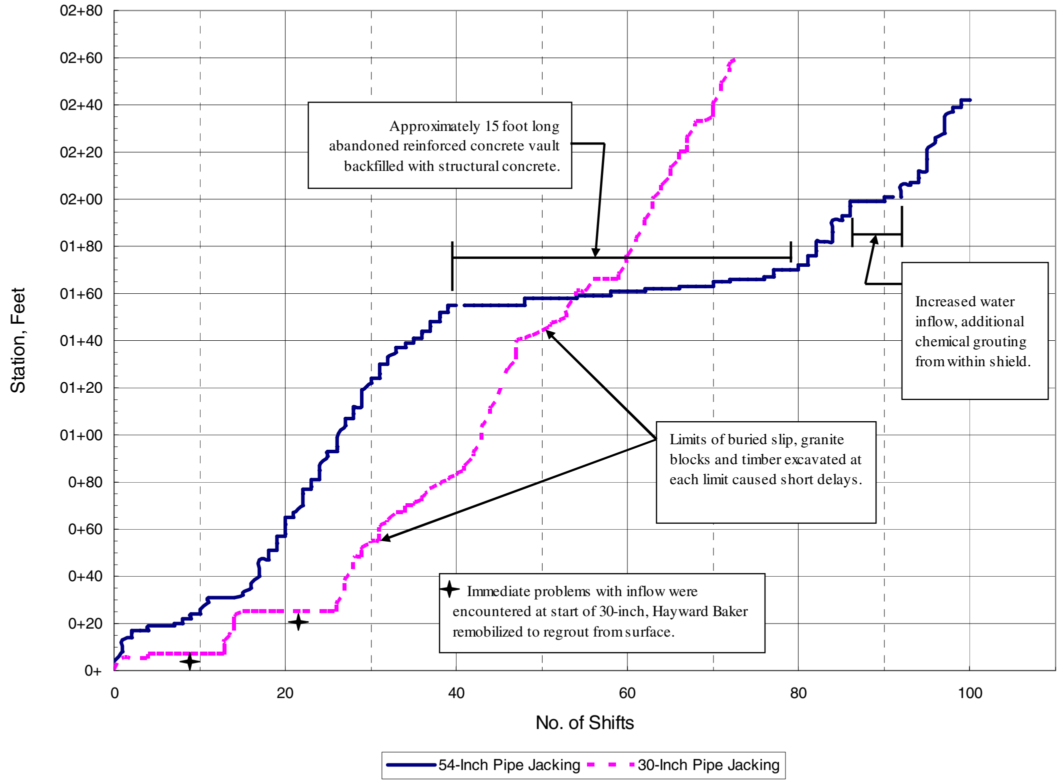

Figure 6. Pipe Jacking Timeline.

Horizontal Directional Drilling With Ductile Iron Pipe

Jami D. Pompeo, EIT(1) – United States Pipe & Foundry Co., LLC

Since its development in 1948, and its introduction by the industry in 1955, Ductile Iron Pipe has become the most widely used product for water line installations. In conjunction with water line installations, Ductile Iron Pipe has become often used for gravity and force main sewer applications. Following a tradition with hundreds of years of history, Ductile Iron Pipe was a logical progression from its predecessor, Cast Iron Pipe. Ductile Iron Pipe has been widely embraced due to its inherent strength, ease of field adaptability, and myriad of joint options.

When the directional drilling method of pipeline installation was first introduced to the United States, members of the ductile iron pipe industry did not view their product as applicable to this method of installation. The reason; it was incorrectly believed that the directional drilling installation method required a pipe material with a uniform outside diameter throughout the entire pipe length. Consequently research and development efforts were put into action to design and test a ductile iron pipe restrained joint with a uniform outside diameter.

During this time of Research and Development by the Ductile Iron Pipe Manufacturers, Horizontal Directional Drilling became a rapidly growing form of pipeline utility installation. There are several key reasons for this installation method growth. Some of the reasons are utility replacement in areas of well maintained landscape, residential areas with well maintained streets and driveways, as well as highly sensitive Historical areas. Open trench construction techniques can be very disruptive and controversial in these sensitive areas. Environmentally sensitive areas have also proven to be key areas where Horizontal Directional Drilling installation methods lessen if not eliminate impact on environmentally sensitive areas.

Ductile Iron restrained joints have been around for decades. These types of joints are proven to be strong, leak free, and flexible. Interviews with directional drilling contractors reveal horizontal directional drilling installations employing ductile iron restrained joint pipe as early as 1992. Many of these installations used standard push-on joint (manufactured in accordance with ANSI/AWWA C111/A21.11) pipe restrained by stainless steel locking segments vulcanized into the rubber gasket. Such a gasket is the FIELD LOK 350® Gasket produced and marketed by United States Pipe and Foundry Company, LLC ("U. S. Pipe"). This type of restrained joint is a great joint, but not for installations that require constant motion such as bridges, pumps, pipe bursting, and horizontal directional drilling. The stainless steel locking segments will fill up with metal under constant motion. Other installations employed individual ductile iron pipe manufacturer's proprietary flexible restrained joints, such as the TR FLEX® Restrained Joint that is also manufactured and marketed by U. S. Pipe, shown in Figure 1. TR FLEX® pipe utilize a push on joint that employs a slightly modified bell, the addition of a high strength stainless steel weld bead on the plain end (19,000lbs/in), and locking segments that butt up against the weld bead. The modified bell allows space to accept the locking segments which during tension allows the locking segments to "lock" against the bell providing positive restraint. It is this type of restrained joint that has evolved into an approved, and commonly used joint type for horizontal directional drilling installation.

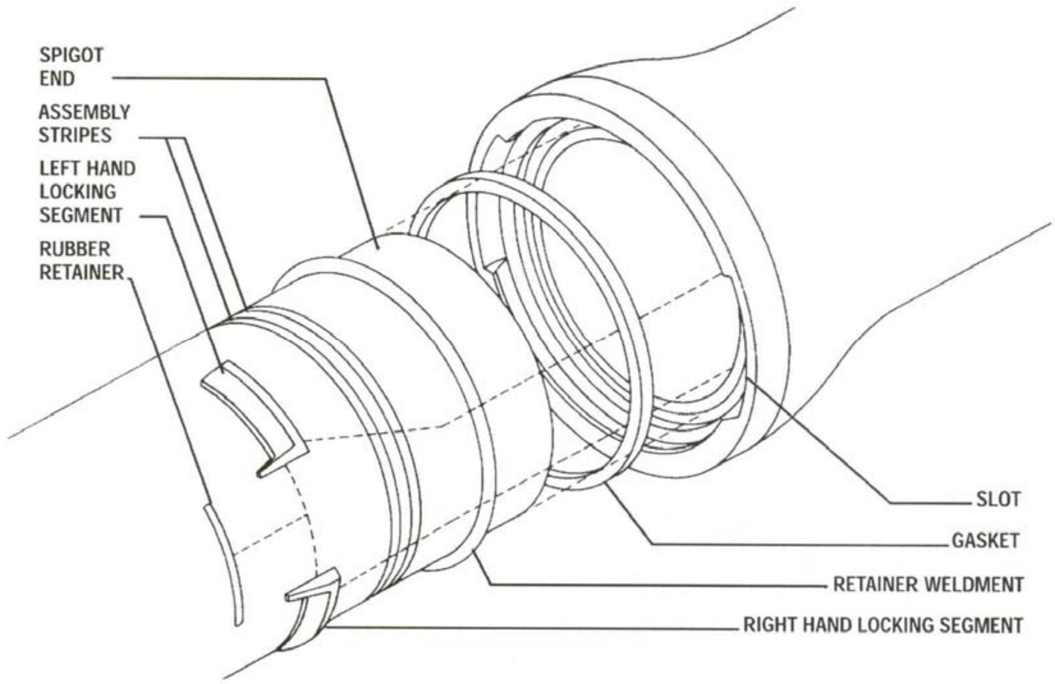

Figure 1. U.S Pipe TR FLEX® Joint(2)

INSTALLATION

Horizontal Directional Drilling involves drilling a bore hole with pilot drill, reaming the bore hole, and pulling the carrier pipe back through. During boring and installation, drilling fluid – water with the addition of bentonite and/or polymers, is injected into the bore head to mix with soil cuttings to form a slurry that supports the bore hole, and gives the new carrier pipe a medium that it may easily pass through. The slurry must have sufficient gel strength to support cut solids suspended in the slurry for evacuation, while at the same time have qualities that enable the formation of filter cake to line the bore hole and maintain structure, while reducing the amount of filtrate (fluid leakage from the bore hole).

The slurry medium that will transport the carrier pipe, being a water based medium will result in the carrier pipe experiencing buoyancy during the pull back section of the drilling process. The bulk density of the Ductile Iron has a more neutral buoyancy more closely resembling the bulk density of the slurry medium. This will result in the Ductile Iron traveling more towards the center of the drill hole experiencing less friction during pull back, where as other material will be more positively buoyant generating friction forces at the top of the bore hole.

The restrained pipe joint must be capable of withstanding the pulling forces applied by the directional drilling machine to pull the pipe through the borehole into the final installed position. It is recommended that the pulling force be limited to the maximum dead end thrust load generated when the pipe joint is pressurized to the joint's rated working water pressure plus a surge allowance of 100 psi as specified in ANSI/AWWA C150/A21.50. Even though Ductile Iron is an inherently stronger material and can withstand higher pulling forces, with Ductile Iron having a more neutral buoyancy causing less friction during pullback it can generate less pulling force required by the drilling machine during pull back. Below is a table showing pull back forces relating to pipe diameter.

End-Pull-Pulling force for TR FLEX® pipe is based on the dead end thrust of the working pressures. Pipe should only be pulled from the spigot, taking advantage of the low profile, streamlined bell. (3)

Pipe Size (inches)	Max. Pulling Force (lbs.)	Pipe Size (inches)	Max. Pulling Force (lbs.)
4	8,143	14	82,734
6	16,827	16	107,004
8	28,947	18	134,391
10	43,546	20	164,896
12	61,581	24	235,256

In 2002, U. S. Pipe personnel monitored a horizontal directional drilling installation of approximately 4700 ft of 12-inch TR FLEX® Restrained Joint pipe with a maximum pulling force allowance of 61,581 lbs. The installation lengths were broken down into a 1,100 ft length, a 1,300 ft length, and a 2,300 ft length. The pulling force for the 1,300 ft length varied between 20,000 lb and 30,000 lbs. While the last 100 ft of pipe was being pulled into the 2,300 ft long installation, the pulling force began to increase above 30,000 lbs, realizing a pulling force of approximately half of the recommended maximum pulling force allowance.

It is recommended that the bore hole be sized to accommodate the diameter of the pipe as well joint deflection required to complete the drill. During a straight pull, where minimum joint deflection is required, it is recommended that the bore hole be sized to 1.25 X the OD (Outside Diameter) of the Bell. During Radius pulls industry standard recommends that the bore hole be sized to 1.5 X the OD of the Bell to accommodate the OD of the pipe as well as limiting the deflection of the pipe joint to one half (1/2) the pipe joints deflection allowing ample room to make the curve. U.S. Pipe and Foundry Co., LLC has developed a radius calculator located at www.uspipe.com that will aide in calculating directional drilling radii per ductile iron pipe diameter. The TR FLEX® joint is recommended for these applications due to the flexibility of the joint, which may require multidirectional deflection that may be encountered during the pull back process. Ductile Iron pipe will not experience curvature stresses during installation. The TR FLEX® locking segments uniformly transfer pulling forces to the bell of the following pipe, alleviating any wall stress on pulled curves. Fused or welded lines experience tensile stress on the outside wall and compressive stress on the inside wall when the pipe is curved. When using Ductile

Iron for Directional Drilling consideration of drill path design is essential in assuring that the joints will not be over deflected.

Using Ductile Iron Pipe for Horizontal Directional Drilling opens up new possibilities, and can save time and money on preparation costs. With Ductile Iron installations, you have the option of cartridge assembly or assembly line installation. The cartridge method involves installing one joint at a time and requires a lot less right-of-way space. Nominal length joint assemblies do not take appreciably longer than the time it takes to disassemble and stack drill rods on the other end. Other materials do not offer this option. For an installation with a limited right of way, this may be the preferred or only option. The assembly line method requires stringing and assembling the joints above ground and ramping the pipe pit to accommodate installation with out over deflecting the joints. For other materials the assembly line installation is required due to the time it takes to fuse or weld the pieces together.

Throughout it's use in the Horizontal Directional Drilling use, extensive research by the Ductile Iron Pipe Research Association and accompanying companies have concluded that the Bentonite mixture for drilling fluids is non corrosive to Ductile Iron, therefore in soils that are not corrosive to Ductile Iron polyethylene encasement is not required. In soils that are corrosive to Ductile Iron, polyethylene encasement is the corrosion protection that to be used during directional drilling installations. Polyethylene encasement is the accepted corrosion protection for ductile iron pipe installations in corrosive conditions. Since approximately 2000, the Ductile Iron Pipe Research Association has observed some restrained joint ductile iron pipe horizontal directional drilling installations with polyethylene-encased pipe. Test holes excavated after the installation was completed revealed the polyethylene encasement to be intact and in place. If the installation encounters stone laden soils, a double layer of polyethylene encasement is recommended. Although polyethylene encasement may not be required in some installations, it may help alleviate some pulling forces. Where hydrocarbons are present in the soils, unlike other material that are pervious to hydrocarbons, Ductile Iron is impervious and will not be effected by the presence of hydrocarbons. Although Ductile Iron will not permeate hydrocarbons, Nitrile or Viton gaskets are required in lieu of the standard Styrene Butadiene Rubber (SBR) gaskets. Viton gaskets are the recommended gaskets for use in hydrocarbons allowing zero permeation of hydrocarbons.

Many of the reasons Ductile Iron Pipe is used for open cut installations are the same reasons it would be desirable for Horizontal Directional Drilling applications. Ductile Iron has up to 20 times the tensile strength of other directional drilling piping materials, 18 times less thermal expansion coefficient, and does not experience tensile creep over time. Ductile Iron will also withstand up to 6 times the hydrostatic burst pressure of other directional drilling piping materials, and incorporates a 100 psi. surge allowance in its design. Additionally, Ductile Iron has up to 82 times the crushing load strength of other directional drilling piping materials. This can be extremely important in Horizontal Directional Drilling applications where the pressure of the drilling fluid / slurry may be increased. The TR Flex® joint can withstand up to 430 p.s.i. external pressure. Ductile Iron has a larger nominal ID, allowing for greater flow or the installation of a smaller diameter line, especially in high pressure applications.

CONCLUSION

Actual installations have determined restrained joint ductile iron pipe to be a viable pipe material for use in the horizontal directional drilling method of pipeline installation. We can see that Ductile Iron Pipe is easy to assemble, field adaptable, and offers the long term strength that end users have come to expect from Ductile Iron Pipe. It's a stronger material that can handle pulling loads, and will not buckle in the hole, even under high slurry pressure. Today's drilling machinery can generate substantial pulling forces on pipe material during Horizontal Directional Drilling installations. Engineers, Municipalities, and Contractors can now rest easy knowing they have a proven, time tested, strong and adaptable pipe material in Ductile Iron that can be used for their Horizontal Directional Drilling applications.

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(2) United States Pipe and Foundry Co., LLC, *TR FLEX® Ductile Iron Pipe and Fittings, 2006 Edition.*

(3) United States Pipe and Foundry Co., LLC, *TR FLEX® Pipe TRENCHLESS APPLICATIONS WITH DUCTILE IRON, Horizontal Directional Drilling (HDD) and Pipe Bursting,* 2006.

Innovative Record Length Twin 60-inch Microtunnel Drives Beneath US 50 and High School in West Sacramento, Combine Direct-Jacked Carrier Pipe and Casing and Carrier in Single Drive

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ABSTRACT

The 19-mile, Lower Northwest Interceptor (LNWI), consists of twin 60-inch and 66-inch forcemains. The LNWI construction required 12 twin microtunneled crossings including a crossing of the River City High School Campus.

When the preliminary design called for the installation of the twin 60-inch steel force mains across the campus by open-cut methods, the School District placed severe restrictions on access to the site including time allowed for construction and restoration. The design engineers (CH2M HILL and Bennett/Staheli Engineers) then recommended tunneling to reduce cost and mitigate access and schedule restrictions.

The contract as bid required two 1,200 foot microtunneled bores across the school campus and two 250 foot bores across a freeway operated by Caltrans from a common jacking shaft located immediately south of the freeway on the High School property. In order to further reduce schedule time, the Contractor proposed to combine the Freeway Tunnel and High School Tunnel into one tunnel push. Caltrans required a casing and two-pass installation but the High School Tunnel was only economical as a single pass tunnel. This required pushing two different pipe types in one tunnel push. It is believed that these twin, 1,420 foot tunnels are one of the longest 60-inch microtunnels completed to date.

In addition to the 1,420 foot drive lengths, some of the additional challenges included microtunneling with shallow cover in close proximity to the high school's swimming pool, tennis courts, and running track. The contractor, Michels Pipeline, was able to complete both tunnels with minimal problems.

Introduction

The River City High School tunnel was one of twelve trenchless crossings along the 19 mile Lower Northwest Interceptor (LNWI). The LNWI is one of several wastewater interceptors being constructed by the Sacramento Regional County Sanitation District (SRCSD) of Sacramento, California and will serve the rapidly growing areas of Northern Sacramento County as well as the City of West Sacramento in Yolo County. The City of West Sacramento's wastewater treatment facility is approaching its design capacity and would have needed extensive modifications to meet new discharge limits being promulgated by the Central Valley Regional Water Quality Control Board. The City approached SRCSD for entry into the District during the preliminary design phase of the LNWI.

Preliminary Design

The entry of West Sacramento into the SRCSD allowed the routing of the interceptor away from downtown Sacramento. The selected route began at a pump station near the intersection of Interstate 5 and Interstate 80, just north of downtown Sacramento. Twin 60-inch diameter force mains were constructed southwesterly, crossing under the Sacramento River and into the City of West Sacramento (see Figure 1).

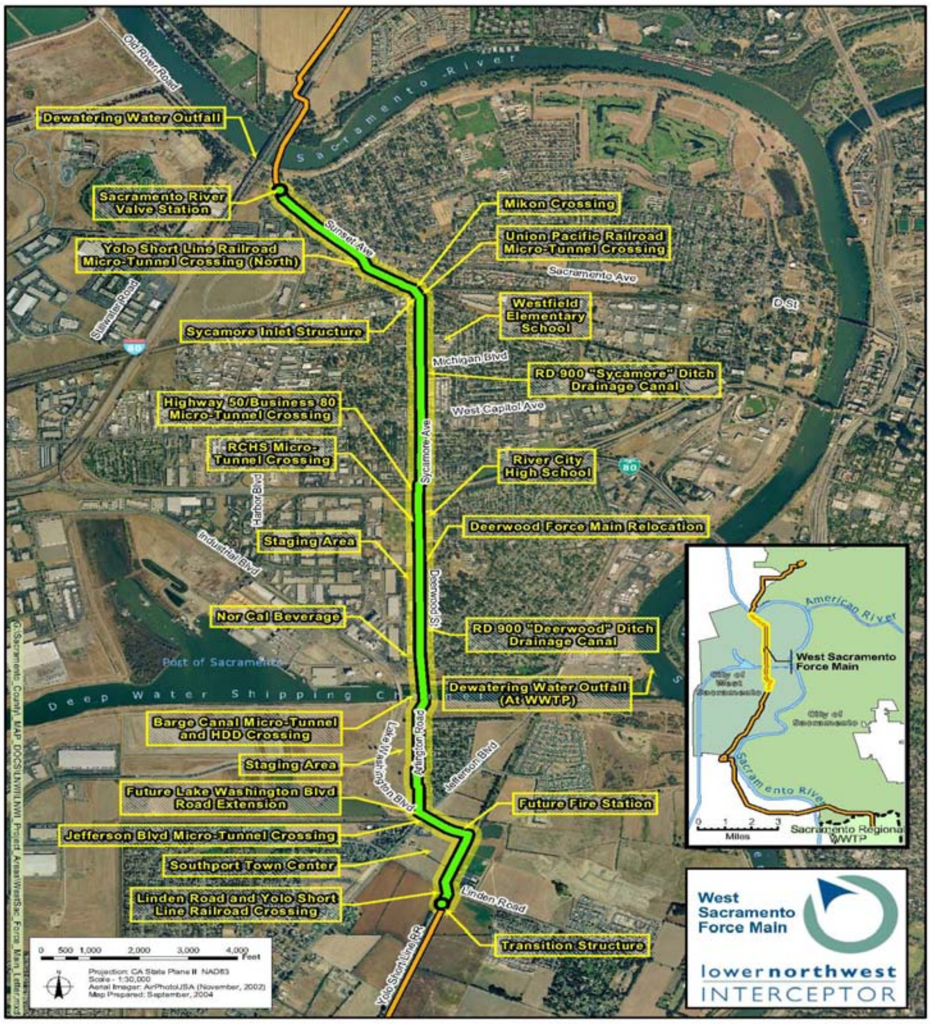

Figure 1 - West Sacramento Force Main

When pipeline alignments through the City of West Sacramento were discussed with City officials, it became obvious route selection would be a problem. Due to the Union Pacific Railroad tracks passing through the City in an east-west orientation, there were only three streets where the interceptor could be constructed and they were already congested with underground utilities. Traffic control and detours would be a nightmare. The City then directed the preliminary interceptor designers to a drainage canal and corridor owned by Reclamation District 900 (RD-900). RD-900 is one of dozens of local jurisdictions in California that were organized to provide flood control, irrigation and drainage services to local farmers, although its irrigation services are no longer needed.

The RD-900 drainage corridor contained drainage canals, however the existing drainage canal could be replaced with a 78-inch storm drain in the northern half of the corridor (Sycamore Drain) and a 60-inch storm drain in the southern corridor (Deerwood Drain). Using the drainage canal corridor allowed approximately 3 miles of the interceptor construction through the heart of the City to be constructed outside the public right of way except for open cut crossings of four streets and a tunneled crossing of US Highway 50. Highway 50 is controlled by the California Department of Transportation (Caltrans) and Caltrans requires pressure pipelines to be encased when crossing the freeway right of way.

However, immediately south of the Sycamore Drain and the Highway 50 Crossing, the force main interceptor must cross the River City High School campus which is owned by the Washington Unified School District (WUSD). The campus crossing would be a simple crossings from a design and construction standpoint compared to the narrow RD-900 corridor in most of the City. But the alignment was politically sensitive since, the City had preferred the RD-900 corridor route, but the School Board did not feel their concerns were heard and properly addressed. The School Board would publicly voice their objections to the sewer construction during the design and construction phases. During the preliminary design it was determined that opencut construction across the athletic fields would be the least costly alignment.

The preliminary design was conducted by MWH Americas. During this stage, the entire 19 mile alignment was divided into seven design contracts which included five pipeline segments, the tunneled Sacramento River Crossings and the design of the two pump stations. The two pump stations are designed for peak capacity of approximately 200 mgd. The 6 mile long, twin 60-inch force main from the New Natomas Pump Station transitions to a 4 mile long 120-inch diameter gravity sewer south of the City of West Sacramento. At the end of the gravity sewer, the South River Pump Station discharges flow to a 9 mile, twin 66-inch force main which connects to the Influent Junction Structure of the Sacramento Regional Wastewater Treatment Plant.

Program Management

Also during the preliminary design, SRCSD determined that growth in the service area of LNWI was developing much faster than expected and the interceptor would need to be designed, constructed and brought on-line in four and one-half years to prevent sewer overflows. MWH Americas was retained by SRCSD to provide Program Management Services and augment District staff to oversee the design services provided by seven other design consultants and five construction management consultants. Approximately $400 million in construction under seven contracts would be under construction simultaneously.

Design

CH2M HILL was selected by SRCSD and the LNWI Program Management to provide design services for the West Sacramento Force Main (WSFM), the force main segment through the City of West Sacramento and across the River City High School Campus. Bennett / Staheli Engineers subcontracted to CH2M HILL to provide microtunneling design services on the project. Due to the design issues and political issues, the WSFM was the most complicated of the projects under the LNWI. It is also considered the most complicated project SRCSD has constructed since the WTP construction and inception of the regional system 25 years ago.

During the 30% design phase, CH2M HILL and Bennett/Staheli studied alternative alignments for crossing the High School campus. Three alignments were considered, including the preliminary design alignment through the athletic fields. Also considered was an open cut alternative that followed the perimeter of the campus and a tunnel alternative. Bennett/Staheli considered another tunnel alignment that started on the north side of the freeway and tunneled the entire school property including the bus maintenance yard. This alternative was not considered further because it was assumed the entire length of the tunnels would be two pass with welded steel pipe inside a steel casing in order to meet the Caltrans requirement of a casing. Since the two pass was not required under the high school grounds, the additional cost for the two pass was a waste of money.

The cost estimates of the three alternatives found the preliminary design alignment through the athletic fields to be the least costly, but that estimate could not accurately estimate the restoration costs and mitigation costs that would be required by the School District for providing a sewer easement. Since schedule was critical on the entire LNWI Program to assure the interceptor was online and functioning by the end of 2006 to prevent possibility of sewer backups or overflows it was decided to proceed with the design of the open cut alignment, but if restoration costs made the open cut alternative more costly or other factors made it infeasible, the tunnel option would be revisited.

When WUSD provided an estimate of the cost to restore the football field, bleachers and other athletic fields totaling over $3 million, CH2M HILL recommended the consideration of tunneling the entire High School Campus. It was also thought much of the issues the School District officials had with the impacts of construction to the community could be avoided by tunneling. For example, it was feared the open cut work would have significant impacts to summer school classes and little league games normally scheduled for the High School field. The School District required construction noise levels to be less than 70 dBA outside the closest classroom which was less than 400 feet from the alignment.

SRCSD agreed and tasked CH2M HILL with analyzing and providing cost estimates of tunnel alternatives for comparison to the open cut method. CH2M HILL and Bennett/Staheli considered four alternatives that included tunneling a portion or the entire campus, as shown in Figure 2. When compared to the open cut alternative, all tunnel alternatives were less costly than open cut plus the surface restoration and mitigation fees. A common jacking shaft for both the Highway and High School crossings provided a significant cost savings. Table 1 presents the comparative estimated costs for the five final design alternatives.

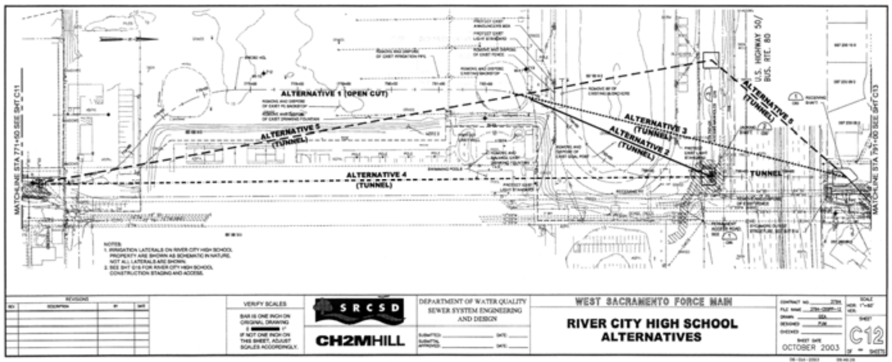

Figure 2 High School Tunnel Crossing Alternatives

Table 1 – High School Crossing Alternatives Including Mitigation Cost

Alt. No.	Tunnel Length (Feet)		Open Cut Length (Feet)	Cons. Cost Estimate	Mitigation Cost Estimate	Total Cost Estimate
	Two-Pass	Single-Pass				
1	288	0	1,432	$3,896,947	$3,180,000	$7,076,947
2	288	400	1,130	$4,134,814	$1,740,000	$5,874,814
3	687		1,130	$4,536,388	$1,740,000	$6,276,338
4	288	1,529	0	$4,977,838	$620,000	$5,597,838
5	374	1,547	0	$5,334,138	$620,000	$5,954,138

SRCSD accepted Alternative 4 as the preferred method of crossing the High School campus and directed CH2M HILL and Bennett/Staheli to prepare plans and specifications for microtunneling the campus.

The main concern for the School District officials with a tunnel approach was the potential for damage to the swimming pool. The top of the microtunnel boring machine would pass about 15-feet below the bottom of the pool. It was agreed that monitoring points would be placed around the edges of the pool deck and surveyed daily to assess settlement while the tunneling was in progress. Weekly surveys were also conducted for 3 months after the completion of the tunnel.

Tunnel Design

The tunnel was designed with three pipe diameters (18-feet) of cover at its lowest point. In order to prevent air entrained in the sewage flow from being trapped and accumulating in the twin force mains, they were designed with a constant upward slope of 0.004. This would also prevent the need to install air release valves and access vaults along the tunnel alignment and within the High School Campus. The more sensitive structures to be crossed which included the freeway and swimming pool were at the deeper end of the tunnel.

Geologic Conditions

Published geologic literature indicates the High School Tunnel project is underlain by undifferentiated Holocene-age alluvial and basin deposits, Pleistocene-age alluvial deposits of the Riverbank and Modesto Formations, and Pliocene-age alluvial deposits of the Laguna Formation.

The High School Tunnel is located almost entirely within the Holocene basin deposits. Basin deposits consisted of dark silt and clay deposited by rivers and streams on natural levees and floodplains. Basin deposits were relatively thin ranging from 6 to 30 feet thick and were underlain by older alluvium. Holocene alluvial deposits consisted of silt, sand and gravel deposited by the present-day river and stream systems including the Sacramento River, in channels, on natural river banks and floodplains.

Soil Conditions

Soil borings were conducted at the launching and receiving shaft locations. A boring was also conducted at the proposed shaft location just south of the freeway as well as intermediate locations between the shafts but just off the alignment.

At the launch shaft location north of the freeway, two soil layers were encountered at the tunnel elevation. The upper half of the tunnel was in a layer of medium dense clayey sand (SC) and the lower half within a medium dense sand with silt layer (SM). Under the freeway, the upper layer encountered by the MTBM transitions to a dense silty sand (SM) and the lower layer to a medium dense to very dense poorly graded sand (SP). As the tunnel progresses to the south, the MTBM will slowly rise out of the dense sand layer, through the silty sand and into a layer of stiff to very stiff lean clay (CL). In addition, there are lenses of sand bedded within clay layers and clay lenses bedded within sandy layers. The entire tunnel was below the groundwater table.

Construction

The West Sacramento Force Main construction project was bid in April of 2004 and awarded to the low bidder, Mountain Cascade of Livermore California in July of that year. Their subcontractor for microtunneling was Michels Pipeline of New Berlin, Wisconsin. The low bid for the entire contract was $60,792,100. Based on the unit price items provided in the bid for the High School and Highway 50 Crossing components, the bid cost of the crossings was approximately $6,846,000.

The contract had a schedule requirement for being substantially complete within 800 calendar days. Due to delays in completing the design, obtaining an Army Corp. 404 permit and obtaining rights-of-ways, the construction contract may not be substantially complete until October 10, 2006. This left very little float in the construction schedule and any construction delays could prevent the startup of the pump stations prior to the December 31, 2006 deadline. MWH was responsible for the overall LNWI Program completion schedule. The MWH Construction Manager asked all contractors for ways to reduce construction schedule. Any contract changes suggested by the contractor that maintained the original design intent, but reduced cost or schedule would be viewed as a Construction Incentive Change Proposal (CICP) and any savings would be shared equally between the owner and the contractor.

Mountain Cascade responded with a proposal that would not significantly reduce contract price, but could save two months of the contract time. Their proposal would change the construction of the High School and Highway 50 crossings by combining them into one pair of tunnels and eliminating the launching shaft on the High School grounds. While the design considered it unpractical due to the perceived need to tunnel the entire length with a casing and twopass system, the Contractor developed a way to push RCCP pipe for the first 1,100 feet and steel casing for the last 300 feet. They proposed using the receiving shaft north of the freeway as the launching shaft, so the steel casing pipe would end up under the freeway when complete to meet Caltrans requirements, but there would be no need to add a carrier pipe under the high school grounds, only under the freeway. The WSP carrier pipe would need to be connected and welded to the end of the RCCP. The Contractor developed an adapter ring for the end of the steel casing pipe so it could push against the RCCP effectively and without damage. A picture of the adapter is shown in Figure 3. The Contractor and Ameron Pipe proposed a joint connection to permanently join the WSP carrier pipe, installed after tunneling, to the direct-jacked RCCP.

Figure 3 - Permalok Adapter Ring

The Contractor's proposal also included moving the receiving shaft at the south end of the High School 200 feet to the north, to a location between the bus yard maintenance area and the tennis courts. When asked, the Contractor said they did not feel confident tunneling the entire 1,700 foot length from north of the freeway and across the school property, since their tunnel boring machine had never been pushed that far and they were not sure if any microtunnel of this size had been successfully pushed that far. Again, the risk of a problem that would delay construction completion was avoided.

The reason the Contractor could offer a schedule reduction was due to the restrictions for constructing the shaft on high school property. The construction contract required the shaft construction to start after the last day of school in June and be completed by the start of school in mid-August. While there was no problem completing the work within that time, the Contractor also had to factor this timing consideration into their schedule for constructing a total of eleven shafts and six tunnel crossings. Each crossing had multiple pipes. Three of the crossings also had inclined tunnels to bring the pipe from deep shafts up to open cut grade. The equipment had to be set up for a total of 19 separate machine pushes. (Although the contractor elected to open cut one of the inclined tunnels.) Since the soil was fairly easy to mine in most locations and 80 to 120 feet was often mined in one day, most of the contractor's time was spent in shaft construction and tunnel equipment set up. If the Contractor's work to construct shafts, set up tunneling equipment and the normal risk of mining caused a slip in schedule, the 2 month window to set up and start the High School tunnel could be missed. It was contractually possible to wait until the summer of 2006 to start the shaft construction, but it would be almost impossible to meet the October completion deadline if that happened. Moving the shaft off the High School property allowed much more flexibility in the Contractor's schedule. As it happened, the Contractor had significant difficulty constructing the shaft for the nearby Barge Canal Crossing. That shaft construction started in December of 2004 and was not complete until Spring of 2006.

The WUSD officials were approached with the Contractor's proposal and they had no significant problems with it, other than the coordination to construct the receiving shaft at the south end of the school campus and the opencut pipeline construction across the bus maintenance yard, which became necessary when the receiving shaft was moved north. The receiving shaft was constructed with steel sheet piles driven with a vibratory hammer. The work was done during the spring break to reduce the noise impact. A plywood fence was built around the shaft. As agreed to with the School, the open cut was completed during the Christmas Holiday vacation break at the end of 2005. The Contractor needed all that time to complete the work due to heavy rains. Some additional work was done to improve and repave the bus maintenance yard to compensate the School District for the coordination they undertook to relocate buses to avoid construction zones.

The launching shaft construction was started in August of 2005 and completed in October 2005. The contractor had problems with running sands and groundwater entering the shaft when an opening for the MTBM launch was prematurely cut in the sheet piles. This was solved by flooding the shaft and contact grouting the outside of the sheet piles. Some ground was lost and settlement occurred outside the shaft and about 20 feet from the freeway sound wall. Survey measurements confirmed there was no movement of the sound wall.

Michels Pipeline used an Akkerman SL74 slurry pressure balanced MTBM to mine in front of the force main pipe. The MTBM and trailing pipe were advanced by a Tabor Minor Company hydraulic jacking unit with four 400 ton cylinders capable of producing a total thrust of 1,600 tons. The contract called for the installation of 60-inch RCCP or 60-inch WSP inside a steel casing where required. Michels proposed using 72-inch ID Permalok steel pipe for the casing which had a 73.5 inch OD. The MTBM was 74-inch in diameter and would produce a 74.5-inch diameter cut. The overcut outside the 72-inch OD RCCP and the Permalok pipe would be filled with a bentonite slurry to prevent settlement and lubricate the pipe string as it passed through the soil.

When not slowed by equipment breakdowns, the Contractor was able to mine between 80 feet and 180 feet per day. The first push was completed in 19 days and the second push required only 13 days. In addition to equipment problems during the first push, the rear shaft wall started to move from the jacking reaction force. The wall was reinforced and soil grouting behind the wall was conducted.

In general the progress was good when tunneling through the sand and slowed when stiff clay was encountered. At times, tunneling progress had to slow because the slurry separation plant could not keep up. Survey of settlement monitoring points on either side of the freeway found no significant movement and no movement of the pavement was noted. Caltrans inspectors visited the site on a regular basis. The depth of cover over the tunnel was 22 feet under the freeway. Figure 4 is an aerial view of the high school campus.

Figure 4 - High School Aerial View

The only significant problem started occurring on the fifth day of tunneling when a slurry frac-out was observed adjacent to the running track and additional frac-outs continued on an almost daily basis. Slurry frac-outs also occurred onto the basketball courts and tennis courts. The basketball court pavement also heaved between six and eight inches. The Contractor cut small holes into the tennis court pavement to relieve slurry pressure under the pavement and prevent heaving. The heaving damaged the basketball courts but did not damage the tennis courts. The Contractor cleaned up all frac-outs and repaired the damaged basketball courts. Figure 5 is a photo of a typical frac-out.

Frac-outs had occurred on a previous tunnel under this contract, so it was not unexpected on the High School crossing, but the number of frac-outs was surprising. Although no definite conclusion was reached, it was theorized that mining from a sand layer to a clay layer caused face pressures to increase quickly, although that was not always observed on pressure gauges in the MTBM control room. In addition, the depth of cover was 15 feet and there was likely several feet of fill over the natural layers of soil being tunneled. These imported soils were less dense and more permeable than the natural soil, allowing an easy path to the surface. Oddly, many of the frac-outs occurred between 50-feet and 100-feet behind the machine face. The MTBM itself was also better suited to sandy soil than clay. Progress occasionally became very slow, but water jets on the mining face usually broke up the clay when that happened. The influence of water jets on the frequencies of the frac-outs is speculative. No frac-outs occurred in the crossings with sandy soil.

Figure 5 - Typical Slurry Frac-out

The MTBM operator had to concentrate on keeping line and grade. The different soil layers had a tendency to let the machine drift and once the machine was off grade, it was very difficult to bring it back to proper grade with several hundred feet of steel pipe connected to the back of the machine. In addition, the second tunnel push had a tendency to drift toward the first tunnel alignment. It was thought the first tunnel disturbed the ground enough to provide less resistance to the MTBM on that side of the face. However, both High School tunneled pipelines were completed within 2-inches of line and grade after tunneling over 1,400 feet.

Conclusion

The total tunnel length of the twin High School Tunnel Crossing was 1,420 feet. The jacking force required never approached the equipment capacity and the intermediate jacking stations were not used. Adjustments to the MTBM face may have improved its ability to mine in stiff clay and reduce frac-outs, but the overall project required mining through both clay and sandy soils. Only minor additional costs were required to repair heave and cleanup the frac-outs. Most importantly, the creative tunneling ideas used for the High School crossing subtracted two months from the Contract schedule. That winter, heavy rains and other construction problems delayed the work, but the schedule float provided by the tunnel change allowed the WSFM project and the LNWI startup to be completed near schedule.

Procedures for Utilizing Vacuum Technology Safely & Effectively

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SUMMARY

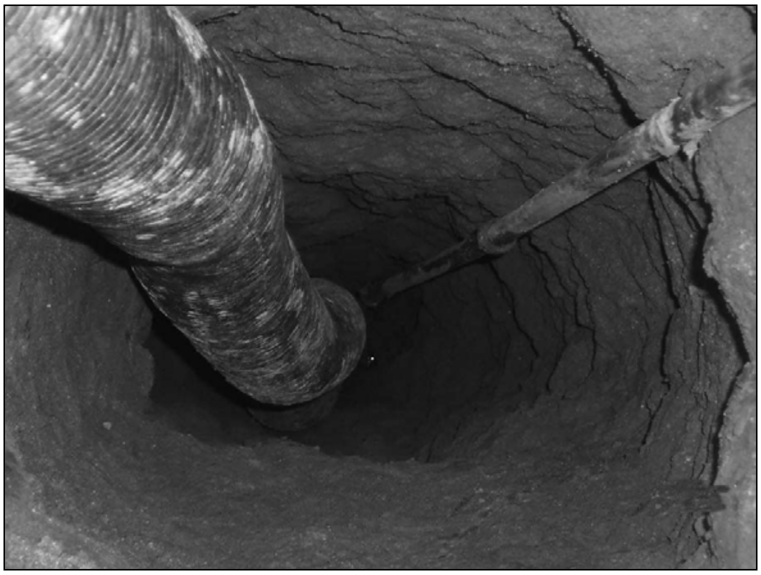
This paper provides an overview of Vacuum Technology and its use in exposing underground utility infrastructures safely and effectively prior to replacement or renewal of aging pipelines. The introduction includes an explanation of why the use of air is preferable over water when utilizing vacuum technology. Applications for use of vacuum technology and the procedures for avoiding conflict are discussed. Step-by-step instructions are provided on how to utilize vacuum technology effectively. Safety standards, conclusions, and acknowledgements complete this overview on the procedures for utilizing vacuum technology safely and effectively.

INTRODUCTION

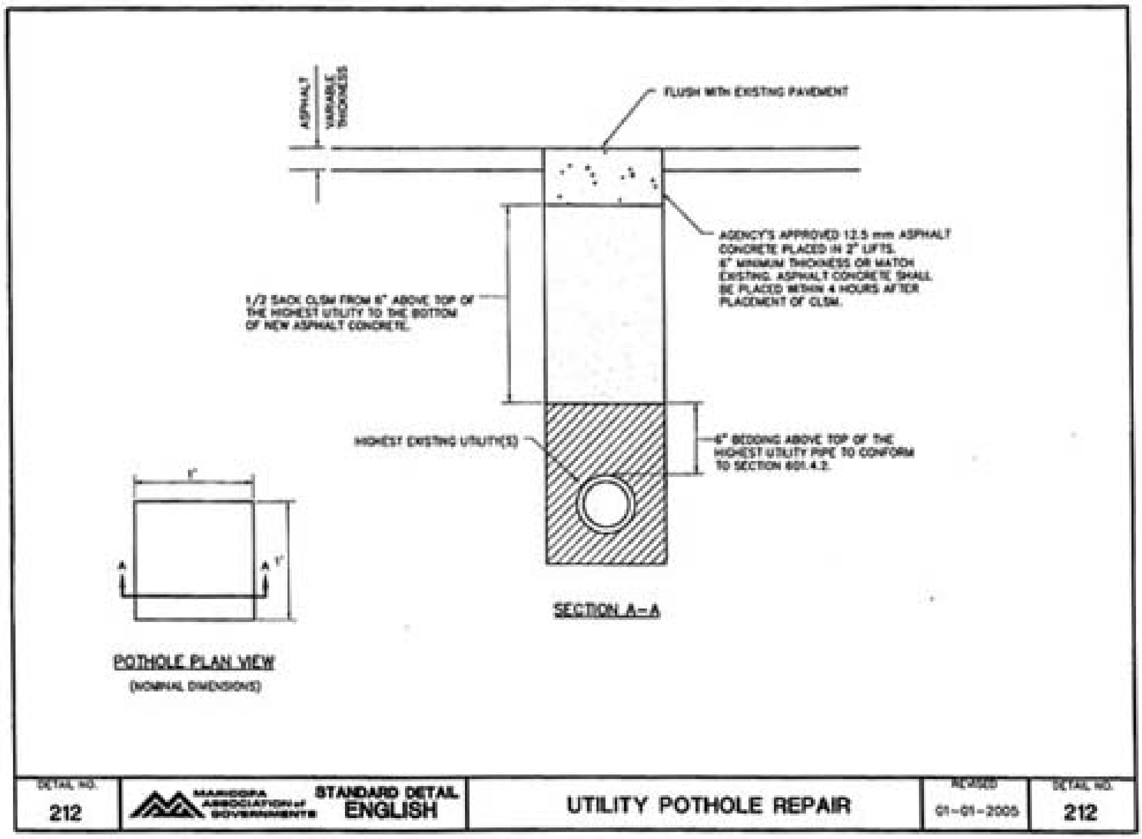
Vacuum technology utilizing air is currently the safest method available for exposing utilities and underground infrastructures. When locating buried utilities, damages to adjacent lines are always a major concern. Often hand digging or vacuum technology are the only options. In recent years, utility regulations often prohibit any mechanized exploration within a few feet of a known pipeline or cable. Vacuum excavation allows contractors to avoid many of these concerns with the least amount of road surface disruption (a big plus in terms of cost and time).

Vacuum Technology is a form of trenchless excavation that utilizes 95% air at a volume of 100+ PSI and 5% water to break up soil in record time. Specialized Services Company (SSC) uses the System 4000, a high-powered vacuum that excavates the material into a 450 gallon rear mounted collection canister. The material is then reused for backfill at the completion of the locating project. Air offers higher performance in most soils, greater efficiency, lower cost, and is less invasive than water. This technology is currently the safest method available to expose underground utilities. The vacuum is 1000 cfm, 15" Hg, with a 4" hose. The compressor has 300 cfm @ 220 psi for the air lance and 300 cfm @ 100 psi for air tools. High pressure water is 0-3000 psi @ 3 gpm and low pressure water is 0-1000 psi for clean-up.

Using an air-vacuum system, you can dig a 12" x 12" x 5' pothole in just 15 minutes in most soils. High-pressure air has many advantages since it is safer than water, the spoils can be used as backfill, and it is faster in a variety of soils.



High-pressure water is not as safe as air, can cut utilities and is a conductor for electricity. Utilizing water as a means of breaking down surface materials creates mud (which does not meet the MAG standards for backfill) and thus requires a need to import backfill. Since the spoils must be hauled away and disposed of, this requires a dumpsite for the excavated materials, which adds to the cost of the project. The standard for backfill is MAG 601.4.3 and COP MAG supplements, which addresses the quality of the backfill, the maximum lift height for the size of the trench, adequacy and uniformity of the compaction. The correct type of backfill must be used for the location of the trench.



MAG 601.4.3

APPLICATIONS

Vacuum excavation is often used when excavating around or near exiting utilities. The kinetic energy of the high velocity air stream penetrates, expands and breaks-up loose soil. The soil and rock are then removed from the hole with a powerful vacuum. Vacuum technology is typically used to create a hole 1' to 2' square and as deep as required to expose the buried utility.

Vacuum excavation is perfect for a variety of other applications, including: utility locating, vault cleaning, manhole and catch basin cleaning, culvert cleaning, debris removal, vertical excavating, water meter box cleanout, landscape rock removal and confined space excavation.

Trenchless technology minimizes the disruption of traffic, reduces noise and dust by providing a smaller and safer work site. There is less need for disturbing the existing environment. Since trenchless technology requires less space underground, it minimizes the chances of interfering with existing utilities or abandoned pipes. Within technology limits, it provides the opportunity to upsize a pipe without open trench construction. The demand for trenchless technology continues to grow as the population increases and existing infrastructures continue to deteriorate as it ages. Since there is a less exposed working area, it also is safer for both workers and the community.

PROJECT EXAMPLE

This project involved a historical structure on the campus at Arizona State University in Tempe, Arizona. SSC used its vacuum excavation trucks as an innovative solution to clearing the crawl space of this historic house. The challenge was working in a limited space and preserving the structural integrity of the foundation for the future ASU Creative Writing House. The vacuum excavation crew used its high-powered vacuum trucks to suck the accumulated debris from the crawl space. Approximately 111 yards of debris was removed, crating a 27” crawl space that could then allow for the reinforcement of the aging foundation.

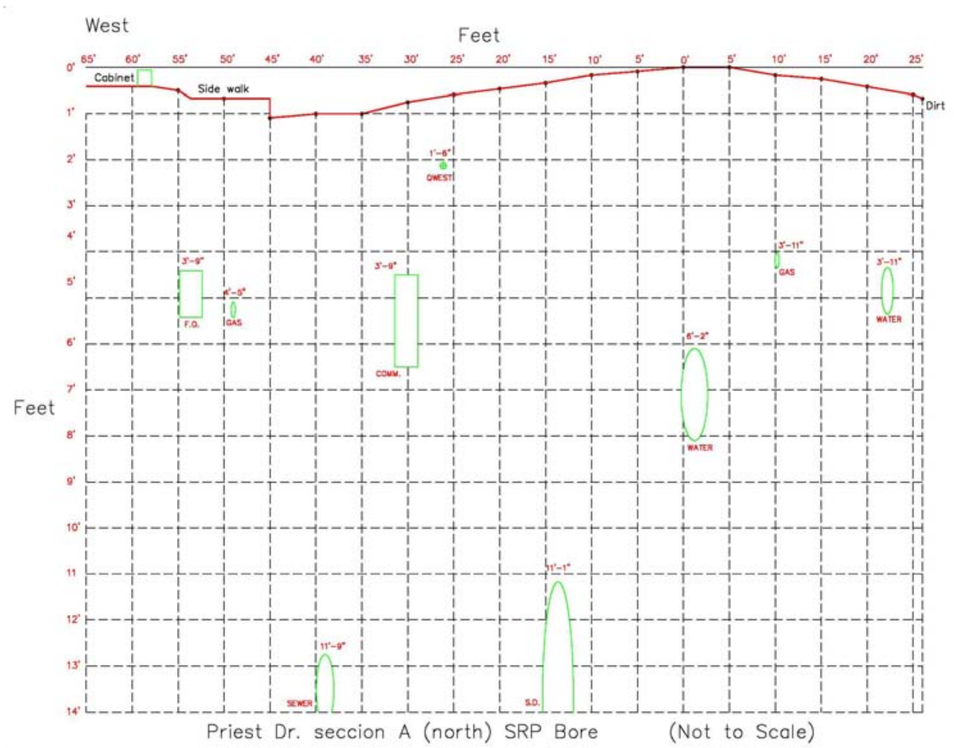
PROCEDURES TO AVOID CONFLICT

Sufficient information is necessary to design an underground construction project. Although traditionally existing records relied upon in the design phase are notoriously inaccurate, contractors, are still by law, responsible to identify, verify, and safely locate all underground utilities at the construction phase. In order to minimize the possibility of litigation and dispute, it is imperative that a contractor go beyond reliance on the ground surface markings of a local one call center and physically perform a surface and often a subsurface survey prior to the dig.

Relying on visual signs in addition to the markings is critical for avoiding conflict. A good surveyor will look for visual clues such as manholes, catch basins, and power lines to detect unmarked conduits. A trained eye can usually detect storm-drains, irrigation lines, private electric and gas conduits, as there are often no evident physical markings. The key is to use trained professionals with knowledge and experience to identify and interpret the clues when performing the survey.

It is also necessary to determine soil conditions and be aware of the actual soil stratifications at a given site. This is accomplished by obtaining soil samples that evaluate the depth and nature of the bedrock, the surface drainage conditions and groundwater levels.

Once underground infrastructures have been identified, a contractor should produce an underground utility profile. This collection of information helps to manage risk associated with placement of future utilities.



STEPS FOR EXCAVATION

1. Secure the site. Use barricades, signs, and other measures to ensure public safety.
2. Break up the surface. A jackhammer may be used initially to break up the surface concrete, asphalt, hard dirt or rock depending on the conditions.
3. Use the air lance. The high pressure breaks up loose soil and creates a pit of the desired depth.
4. Vacuum. Vacuum the loose soil and safely uncover the utility. Vacuum up the dry spoils for easy backfilling. Remember that this eliminates the time and expense associated with mud disposal.
5. Backfill, Compact and Cold Patch. The same material that is removed from the excavation site can be reused as backfill. This meets MAG standards for backfill.

SAFETY STANDARDS

Compliance with all OSHA safety standards is of utmost importance when using vacuum technology. This includes securing the site with barricades, signs, and other measures to ensure public safety.

The standard guidelines for the collection and depiction of existing subsurface utility data (the ASCE Standard) presents a system for classifying the quality of existing subsurface utility data. This allows the project owner, engineer, and constructor to develop strategies to reduce risk, or at minimum, to allocate risk due to existing subsurface utilities in a defined manner. The ASCE standard assists engineers, owners, and contractors in understanding utility level classifications and their allocations of risk.

Using a qualified and reliable contractor is a key component when planning projects involving “marked” and “unmarked” underground infrastructures.

CONCLUSIONS

Vacuum technology for excavating provides a practical, cost effective solution for utility locating without sacrificing safety. Through the use of the right equipment, resources and knowledge of vacuum technology, make your next design-build or construction project a success. Make trenchless technology your first choice.

ACKNOWLEDGEMENTS

For more information regarding the use of trenchless technology, the following websites give further details regarding information found in this abstract.

(1) www.vacmasters.com/airstm.htm from VACMASTERS potholing – underground utility locating website.

(2) www.trenchlessonline.com from article in August 2006 Trenchless Technology Special Supplement, P-7.

CONSTRUCTABILITY OF LARGE DIAMETER PIPELINES IN AN URBAN ENVIRONMENT: A CASE STUDY

INTRODUCTION

The City of Baltimore (City), Maryland has recently undertaken an extensive program to upgrade and rehabilitate their aging sewer system. The City, whose aging infrastructure has led to the necessity to replace and upgrade their existing systems, is under an Administrative Consent Order (ACO) to bring their system into compliance with state and federal regulations and to cease polluting the Chesapeake Bay. As part of the ACO, the City is required to upgrade the force main servicing the New Jones Falls River Basin. The work to be performed includes, in part, upgrading the existing pump station, upgrading the gravity sewer, and replacing the existing 36" force main. The replacement of the 36" diameter force main requires the installation of 42", 48" and 54" Prestressed Concrete Cylinder Pipe through city streets at depths well in excess of 20 feet and, in some instances, in excess of 30 feet.

In order to complete this upgrade, the pipelines that are required to provide the City with sewer services are both larger in diameter and more difficult to construct than the existing ones. What is more, the presence of existing utilities makes it more difficult for engineers to find non-conflicting routes for the new pipelines. As a result, the new pipe lines are requiring extensive relocation of existing utilities; they are being constructed deeper and are becoming more costly to the owner.

This paper will follow the New Jones Falls Force Main Project through the following phases: Design; Bidding; and Construction; with the main emphasis on the Construction Phase.

DESIGN PHASE

Due to the difficulty in selecting routes for the pipe line, the design phase took two years to complete. During this phase, several different routes were evaluated. The route that was ultimately selected required extensive relocating of utilities, changes in the pipe alignment within the street right of way and excavations at considerable depths. The need to both change the alignment within the street right of way, and to move the pipe line from one side of the street to the other while avoiding existing utilities and underground structures made the task of installing the pipe line more difficult. To further complicate matters, the streets where the pipes were to be installed were lined with brownstone houses. The presence of these row houses severely limited the size of the construction equipment that could be used, as well as the ability of the equipment to maneuver while installing the pipe.

After two long years of revision, the final agreed upon design called for the open cut installation of approximately 19,400 feet of large diameter force main at depths that, in some instances, exceeded 30 feet while maintaining one to two lanes of traffic. The final design also required tunneling a total of 1010 feet of force main in 13 different locations. Each of these challenges required means, methods, and techniques that were often incompatible with either open cut excavations, the restrictions posed by the city streets, or both.

BIDDING PHASE

The bidding phase of a contract is, generally speaking, short in duration compared to the design phase. A contractor has several weeks at most to fully comprehend and evaluate a project. In order to expedite the bidding process, the contractor will assume that the plans and specifications prepared by the design professional are constructible as shown in the design.

In preparing his bid, the contractor must visit the site, estimate the quantity of work to be performed, obtain pricing of the materials, seek out and procure subcontractors and finally prepare his proposal. Given the variety of tasks that are part of the bidding process, much of this legwork is performed by junior staff members. It is not until just prior to the submission of the proposal to the owner that senior members of the staff and the contractor review the work of their assistants. As a result, it is usually not until a relatively late stage of the bidding process that construction problems are identified by the contractor and discussed with his staff. All too often, by the time the construction problems are identified it is too late to seek clarification from the owner because it is either barred by the language in the "Instruction to Bidders" section of the specifications, or because insufficient time is available to advise the owner of the contractor's concerns.

In a situation where problems are identified and cannot be addressed, a contractor who reviews a bid proposal prepared by his assistants just before the bid goes off, finds himself faced with the difficult decision of whether or not to go through with bidding the project. In some instances, a contractor will bid the project using pricing that covers his costs for the worst possible conditions. In other instances, the contractor will ignore the worst case scenario and assume that the design engineer must have considered these problems and the project is constructible. In most instances, however, the contractor will place a contingency cost in his bid that lies somewhere in between the two extremes. In the case of New Jones Falls, the contractor adopted several different strategies for the anticipated problems. In some of the areas, the contractor compared the cost for open cut versus tunneling and found that they were comparable. In these instances, he priced the open cut excavation for tunneling. These areas are addressed later in the paper and were handled through a no cost change order to the City. In the other less difficult areas, the contractor assumed that the pipe line could be constructed as designed.

In a perfect world, the contractor who bids a difficult project such as the one outlined above using the engineer's design and/or a contingency cost will both get the job and be able to complete the job as proposed. More often than not, however, the ultimate consequence of a scenario such as this is that a contract is entered into with a contractor to perform a project that might not be constructible. In this situation, the contractor is now in control of the project with respect to performance. The ramifications to the owner can range from no additional costs (depending upon the philosophy of the contractor) to either increased construction costs through change orders or, in the worst case, litigation. Fortunately for the City, this has not been the case in New Jones Falls.

CONSTRUCTION PLANNING PHASE

Once the job has been awarded and a contract is entered into, the contractor can begin the planning phase of the project. In this phase, he is expected to evaluate the project and make his final decisions with respect to the means, methods and techniques that he will employ. For the New Jones Falls Project, the installation of large diameter pipe at these depths and conditions required means, methods and techniques that were not always compatible with open cut excavations in restricted city streets.

At New Jones Falls, a laying schedule was prepared for prestressed cylinder pipe that took into consideration the lengths of pipe that would be used to negotiate under utilities and through intersections. This schedule also reflected the consideration of the increased length of the pipe runs that might occur as a result of “creep” through the pipe joints.

Creep occurs because of the tolerances within the pipe joints. There is approximately 2 inches of tolerance in the joint of large diameter pipe. If the pipe is not driven all the way home, but is still driven home sufficiently to ensure a watertight joint, the laying length of that pipe could increase by as much as two inches. If the contractor does not plan for creep, the pipe line can increase in length by as much as two inches for each pipe installed between bends and manholes. The consequences are that in a run of one thousand feet with twenty foot lengths of pipe, the creep could add as much as 100 inches or approximately eight feet to the stationing. This would mean that the pipe line would miss the location of the bend or manhole by as much as eight feet. In city streets or where critical changes of the pipe line alignment must occur, this could be disastrous. To plan for this, contractors will incorporate random lengths of short pipe in their laying schedule to negate the effect of creep and attempt to meet stationing requirements of bends, special fittings and structures.

For the New Jones Falls Project, several off sets (or bends) were used by the design engineer to navigate the pipe line around existing utilities and underground structures. In each instance, the contractor was required to install a bend and either move the pipe line from one side of the street to another or install a bend to cross an existing major water main of 30 inch diameter or larger or sanitary force main of 30 inch diameter or larger. In each case, the location and stationing of the bend was critical to the success of the installation of the pipe. The requirement to meet the exact stationing criteria was tantamount to threading the proverbial needle.

During the planning phase of the New Jones Falls contract, the contractor evaluated the sections of the project that required threading the needle - the use of steel sheeting or “H” piles and lagging to protect the workers and brace the excavations, the requirements to maintain traffic, and those sections that required special back fill considerations to protect existing utilities - before comparing the cost of open cut excavation against the cost of tunneling those sections. It was found that the cost to tunnel was comparable to the cost to open cut. In addition, the advantages to the public by constructing the pipe line by tunneling far outweighed those of open cutting. Generally speaking, tunneling is the most cost effective method for pipe line construction that requires excavations in excess of 25' deep; that are in close proximity to existing structures; that require extensive bracing and the added burden to maintain traffic.

CONSTRUCTION PHASE

Construction on the Jones Falls Project began in May 2006. During the construction phase, it became increasingly evident that the open cut installation of large diameter pipes would require construction equipment and techniques that were not always conducive to the public's best interests. This was largely due to the two-fold problem of accommodating necessary construction equipment and sheeting systems in what is, essentially, an urban environment.

As an example of the considerations given toward the constructability of a project under these circumstances, the equipment that is required to excavate and install pipe is in excess of 80 tons. What is more, the outside width of the tracks of an excavator this size is approximately 15' and the excavator requires approximately 7 additional feet during its swing to accommodate the counter weight. As a result, maintaining traffic on city streets with this size equipment is almost impossible as it poses a danger to both the workers and the public.

Another consideration is that an excavation that is in excess of 20 feet deep in city streets for the installation of a 54" diameter force main will require the use of a sheeting system. The excavation will have an outside edge of sheeting to edge of sheeting of a minimum width of approximately 14 feet. When working in the vicinity of existing utilities, very often the footprint of the sheeted excavation is too large to install the pipe without interfering with the existing utilities. To complicate matters, if a bend is required, the job site footprint will become even larger because the excavator will have to maneuver around the excavation in order to dig the trench

After recognizing and presenting the inherent problems with installing the pipe to the City of Baltimore and its representatives, the City found it to their benefit to enter into a no cost change order with the contractor that would allow the contractor to tunnel portions of the project that were previously designed for open cut. In the areas that it was not economically feasible to use a no cost change order, the City recognized the problems associated with open cut installation, negotiated a tunneling price with the contractor and agreed to pay the contractor for the increased cost to tunnel.

As a result of the City's proactive administration of the contract, the project has moved forward and difficult situations have been addressed to the satisfaction of the City, the public, and the contractor. At the time of this paper, the contract is more than fifty per cent complete and three other sections of the project are being considered for tunneling. The total length of tunneling has been increased from the original 1010' to approximately 4,700'. Furthermore, the sections of the project that were initially changed to tunneling have provided minimal impact to the public and the environment.

Through the cooperation of the City and the contractor, the problems with the constructability of the pipe line were amicably resolved. The tunneling and open cut installations are proceeding on schedule and, barring any unforeseen circumstances, the project will be completed without any cost overruns to the City.

CONCLUSION

In conclusion, the design for the installation of large diameter pipe lines within streets in an urban environment requires the review of experienced construction engineers. All too often, the design engineer believes that the solution to construction problems in city streets is to use either a sheet piling system or smaller equipment. Experience has demonstrated that these solutions do not take into consideration either the increased footprint area of the excavation for installing a sheet piling system with the associated bracing or the need to use cranes and additional equipment to install the pipe when smaller equipment is requested. The ramifications of such an oversight could result in increased costs to the owner and/or litigation. Fortunately, due to the professional experiences of the construction engineer assigned to this project, there were no negative impacts to either the City or contractor on the New Jones Falls Project.

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Sound Baseline Geotechnical Investigation and Interpretation Offers Most Valuable Liability Management in Pipeline Projects

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Abstract

The first author has been practicing geotechnical engineering for primarily pipeline engineering projects for over 30 years and has amassed significant experience under varying site conditions from over 300 projects. The second author has been a contractor-engineer for over 40 years and has built over 200 pipeline projects. Both authors have seen many pipeline projects by either trenchless or open cut construction methods run into serious problems, disputes, arbitrations, mediations, and court proceedings when the engineer of record of the pipeline project failed to do an adequate baseline geotechnical investigation. Often the trench design, dewatering systems, shoring systems, soil support systems, bedding and backfill were inadequate due to the engineer of record not doing an adequate geotechnical analysis and existing utility mapping. These short-comings may result from too much emphasis on profit, too little time to perform all of the steps, or engineering firms allocating budgets that restrict quality engineering of the projects. The pipeline project may also fail due to soil migration. This is a common occurrence when no filter criteria are checked and when no suitable material is designed in the trench zone. Thus, the pipe loses its lateral support, becomes over-deflected, fails to meet the design criteria and in some cases even collapses. Although means and methods are the responsibility of the contractors, licensed engineering professionals simply cannot expect the contractors to construct projects properly when there are too many deficiencies in the engineers' designs, specifications, and mappings of existing utilities. It is not possible for contractors to perform miracles in the field when the engineers of record simply do not afford the standard of care expected of them by their clients. Unnecessary problems can occur when the engineering firm's revenue generation, company growth, and client relations take precedence over staffing pipeline projects with

sufficiently qualified engineers. In summary, the success of the pipeline project depends heavily on knowing enough about what is in the ground, how ground conditions behave toward buried pipelines, and how to cope with difficult site conditions. The authors document some guidelines on the above issues using several case histories from their involvement in over 500 pipeline projects.

Background

The authors came across the following situations on pipeline projects and are compelled to write this paper to help our industry avoid similar occurrences:

- A drainage and outfall project with large diameter HDPE became severely over-deflected and failed to meet the state DOT and city's specifications due to the engineer of record using an inadequate baseline geotechnical investigation and interpretation, specifying a trench design that was meant for reinforced concrete pipe, not including adequate dewatering specifications, not providing clear compaction test specifications, and not providing proper construction oversight. The contractor did not adequately dewater the site during pipe installation, and did not follow the city's specifications. The design engineer also insisted on using heavy reinforced concrete pipe weighing over 1,510 kg/m (1,013 pounds/ft) for the 1220 mm (48 inch) outfall even after the geotechnical consultant reported that the standard split spoon sampler sank into the poor native soils under its own weight before the 64 kg (140 pound) hammer hit the stem. The unfortunate owner looked into relining the new pipe using CIPP even before the remaining portions of the pipeline were completed.

- A 2133 to 2743 mm (84 to 108 inch) welded steel pipe became severely deflected at a $160-million dollar water filtration plant due to improper backfill and bedding selection, improper compaction test specifications, inadequate monitoring of the contractor's workmanship during construction, and the contractor was permitted to remove the stulls too early. Also the engineer of record never provided proper guidance to the contractor to prevent loading the crown of the pipe with heavy construction equipment. The contractor was asked by the engineer to excavate the pipe to correct the extra deflection, thus damaging the steel pipe further. The combination of errors delayed the new treatment plant by more than a year. There was no proper analysis of the geotechnical conditions in this project by the engineer of record.

- Prestressed concrete pipe (PCCP) used in a river outfall could not hold line and grade during installation due to the geotechnical engineer improperly plotting the borehole profile along the pipe alignment at the river crossing. The engineer also mistakenly represented that the bore was in a rock strata when indeed the contractor found weak native soils. Not checking for soil migration and specifying the wrong trenchfill material resulted in a change order of almost $1-million to the owner on a project that was originally contracted for $ 2.5-million.

- Fiberglass pipe was designed for a sanitary sewer without much consideration of other pipe materials. The soils samples indicated organic material in the foundation for about 10% of the alignment. The design engineer recommended to the owner that steel sheeting be driven for the trench wall and be left in place for the life of the project, increasing the cost of the project from $1 to $5-million. The owner who paid design fees for sound engineering decisions had to tell the engineer that this would not be an acceptable design or budget.

- An underwater inverted siphon of 1.83 m (6 ft) diameter to transmit water was designed with only one pipe material lacking any serious consideration of alternate pipe materials. It was supposed to be placed in a new tunnel constructed of 3.66 m (12 ft) diameter tunnel with the annulus to be filled with an unprecedented amount of grout. No consideration was given by the design engineer of sharing such an outrageously large tunnel with other utilities.

- When the primary yard piping for a major water treatment plant was designed with welded steel and PCCP, the consideration for corrosion was solely based on U.S. Department of Agriculture's Soil Conservation Survey maps. The owner told the engineer of record that these maps were outdated, were prepared for crop cultivation purposes, and were never intended for engineering buried pipelines. Due to the engineer of record not having anyone qualified to design the pipelines specified concrete encasement for all of the steel piping. The design engineer did not know how to design aboveground piping on supports, so he chose to bury most of the plant piping beneath the footprint of the water treatment facility. There was no consideration about how any repair work on the buried pipe would be performed during the life of the project should an emergency situation ever arises.

- On a major PCCP rehabilitation project, the design engineer neither had any prior experience with trenchless liners, nor spent any time reviewing any of the data provided by the vendors. He recommended to the owner that the project be bid with an unsuitable liner material and demanded that all other pipe vendors refrain from contacting the owner about his decision. The design engineer did not consider any geotechnical data in the design.

- On one pipeline the project engineer did not coordinate the location of the pipeline in the street with the public works director, so during construction the street owner objected to the location. Moving the alignment forced the contractor to remove and haul the excavated material around the block for backfill. It also reduced the amount of room due to traffic control, and there was no room to store bedding or materials. Making a better investigation prior to drafting the plans, including collaboration with all the affected parties would have saved a lot of money, and shortened the length of the project.

- A river crossing was designed, bid, and awarded to a contractor without any incidents. The engineer had taken on more responsibility for management to increase fees, with one of the responsibilities being permits. When the contractor got ready to cross the river, the river authority stopped the work because the engineer did not have the experience to manage the permit process. This cost a substantial amount of money and took over double the time anticipated.

- A project was awarded to a contractor with gas, sewer, and water pipelines in the same ditch. An inexperienced engineer designed all of the pipelines, along a city street with multiple utilities crossing the new pipeline right-of-way. The contractor was aware that the project was not constructible, so brought his experience into play, coordinated with the engineer and owner, and together was able to avoid a catastrophe.

- In a major city in a downtown congested area, a pipeline relocation project was let without locating all existing pipelines that could have been located during design. The fast track project went into the slow mode when a variety of unanticipated utilities conflicted with the proposed relocation route. The time and resources were not allocated to properly determine the underground conditions during design. It probably cost four times as much as it would have with a much smaller investment in soil exploration up front.

- Many engineers and owners do not understand the economic sense it makes to locate and map new and existing utilities. A new pipeline project installed amidst existing utilities can cost twice as much if the contractor does not know where the existing utilities are. This also applies to the costs of an owner's maintenance crews repairing a pipeline. This does not factor in safety issues, disruption of other utility operators, and the emergencies that develop when there are "hits" to existing utilities because insufficient time and money were allocated to knowing what the job entailed with quality engineering. Such deficiencies have led to innocent construction workers on new pipelines being burned to death in highly congested utility corridors during construction.

The above examples are occurrences among engineering firms that have staff qualified in process engineering, but not geotechnical or pipeline engineering. Yet they are involved in infrastructure projects, but the assigned project manager has neither formal training nor sufficient project experience in either geotechnical engineering or pipeline engineering. These unfortunate outcomes occur with engineering firms that place a heavy emphasis on sales, marketing, and client relations, but fail to recruit and retain senior underground pipeline professionals with adequate geotechnical engineering experience. Often the pipe vendors, who are good sales people, but often lack an engineering background, bring the technical know-how to these complex projects, while they do have a conflict of interest, and don't always consider all of the technical aspects of the job. So a weak engineer might use the manufacturer's data, not use good engineering analysis, and get the owner in trouble Bad projects that cost the public dearly and do not provide the promised results, can lead to distrust of the civil engineering profession like is now prevalent in the electrical utility industry.

Importance of Soil in Pipe-Soil System Design

Underground pipelines can fail in one or more of the following modes:

- Seam separation due to excessive ring compressive forces
- Wall crushing due to excessive stresses
- Buckling due to excessive external pressure or internal vacuum
- Excessive deflection leading to leaking joints
- Excessive total stress or strain leading to local yield
- Excessive deflection during handling, installation, and construction
- Electrolysis from currents in the soils
- Corrosion from chemicals in the soils

The surrounding soil is usually expected to enhance the ability of the underground pipelines to avoid failure in any one or more of the above modes. In this regard, it is the responsibility of the engineer to pay significant attention to the properties of native and trench soils proposed for a given pipeline project, including the following:

- Soil type(s) and their engineering properties
- General bedding and backfill requirements
- Extent of undesirable material(s)
- Chemicals in the soils
- Location of the water table

The pipeline may be constructed by cut and cover, microtunneling, tunneling, boring, casing, or horizontal directional drilling. This should be considered in developing the site investigation program. The following sections provide some guidance on obtaining important aspects of soil behavior needed to successfully complete the selection of pipe materials, backfill, bedding, and installation method, design of the pipe-soil system, and the eventual installation and inspection of underground pipelines.

Challenges of the Underground

The underground poses some formidable challenges to the geotechnical-pipeline design teams. Some of these challenges are as follows:

- The vast uncertainty that is common to underground projects.

- The cost and feasibility of the project is dependent on geology/site geotechnical conditions.
- Some features of investigation for a pipeline project may be more demanding than traditional foundation engineering projects.
- The regional geology must be known.
- Engineering properties change with a wide range of conditions, such as time, season, rate and direction of loading, etc.-sometimes drastically.
- Groundwater is the most difficult condition/parameter to predict and the most troublesome during construction of pipelines.
- Even comprehensive exploration programs recover a relatively minuscule drill core volume, less than 0.0005% of the excavated volume of the project.
- There is no guarantee that the actual stratigraphy, groundwater flow, and behavior encountered during construction may be as per the geotechnical team's predictions.
- Measuring the electrical currents in the pipe zone.
- The chemical properties of the soils for corrosion studies are needed.
- The cost of pipe zone materials, such as bedding.

Geotechnical Baseline Investigation

A good Geotechnical exploration program would help evaluate the feasibility, safety, design, and economics of a pipeline project by the following ways:

- Developing sufficient understanding of regional geology and hydrogeology for project design and construction.
- Defining the physical characteristics of the materials that will govern the behavior of the pipe-soil system.
- Helping define the feasibility of the project and alerting the engineer and contractor to conditions that may arise during construction for the preparation of contingency plans.
- Providing data for selecting alternative excavation and support methods and, where project status permits, determining the most economical alignment and depth.
- Providing specific rock, soil, and hydrogeologic design parameters.
- Selection of construction method: open cut vs. tunneling / pipe jacking / micro tunneling.
- Minimizing uncertainties of physical conditions for the constructor.
- Predicting how the ground and groundwater will behave when excavated and supported by various methods.
- Establishing a definitive design condition (geotechnical basis for the bid) so a "changed condition" can be fairly determined and administered during construction.
- Improving the safety of the work.

- When project funds permit, providing experience working with the specific ground at the project site through large-scale tests or test explorations. This in turn will improve the quality of design and field decisions made during construction.
- Providing specific data needed to support the preparation of cost, productivity, and schedule estimates for design decisions, and for cost estimates by the owner and bidders.
- Analyzing more than one pipeline route if possible.

Additional geotechnical field investigations are generally recommended to:

- Confirm soil boundaries;
- Confirm modes of deposition of overburden;
- Assess potential for boulders, size and frequency;
- Assess potential for plastic clays or hard fissured clays;
- Assess potential for cemented layers within otherwise uncemented strata;
- Evaluate soils of varying strength, cohesion, and water content;
- Evaluate potential for perched or artisan groundwater conditions.
- Confirm that the pipeline route is feasible given all of the other conditions.

The engineer should require that some soil borings be made available in order to begin the design of the underground pipeline project. The borehole and its log should be sufficiently deep in the native soils to indicate the location of the water table and its variation during the year, and the properties of various soils at site in the region of interest. Either split spoon sampler with blow count readings or Shelby tube sampler with undrained shear strength readings should be given on these boring logs. The classification symbol, unit weight, grain size curve, and Atterberg limits should also be given with these boring logs on native soils. The following are some recommendations for developing a field boring program for a typical pipeline installation project:

- For short road, railroad, creek and slough crossings, a minimum of two holes on either end, with preferably a third at an intermediate location, if there is access;
- For alignments of greater lengths, borings at probable turn-points in the alignment, and at 45.7 to 61 m(150 to 200 ft) spacing elsewhere with sampling from one diameter above to one diameter below pipeline/underground opening;
- For pipe jacking/tunnel/microtunnels, borings at shaft locations, to a depth at least one shaft diameter if pit is circular or the length if pit is rectangular or 4.6m(15 feet) whichever is greater below the design invert level, with sampling for full depth of hole;
- Borings at intermediate jacking points for microtunneling, minimum of 2 pipe diameters below lowest invert elevation. Sampling one diameter above to one diameter below underground opening.

- If there is some uncertainty about the exact invert elevation of the conduit at the time of boring the sampling range needs to be extended to cover probable range of invert levels.
- At locations where the topography changes, such as streams, rapid changes in elevation, or at each point where the elevation of the pipeline changes by the depth of the pipeline.
- Where there is evidence of buried materials or signs of abandoned underground structures.
- At locations where it is probable that contaminated soils or hazardous wastes are present.

Need for Laboratory Tests

Laboratory tests on samples need to be performed to define/evaluate:

- Soil behavior at the soil-pipe interface or tunnel/microtunnel face;
- Behavior of subsurface;
- Appropriate means of excavating and supporting underground openings;
- Effectiveness of slurry shield machine in excavating and removing spoils through hydraulic means;
- Need for introduction of water, water/bentonite, and other water/clay mixtures to facilitate slurry make-up;
- Most appropriate means to separate the spoils from slurry systems;
- Need to introduce bentonite to reduce adhesion or friction along the pipe string or steel casing;
- Jacking thrust capacity, maximum spacing for jacking and receiving shafts, and maximum distances before intermediate jacking stations.
- Chemical properties of the soils.

The samples that are obtained during the field explorations need to be examined in a geotechnical laboratory. The physical characteristics of the samples must be noted down and the field classifications modified where necessary. During the course of the examination, representative samples are then selected for further testing. The testing program on the soil samples must include standard classification tests, which consists of visual examination, moisture contents, Atterberg limits, unit weight measurements, and grain-size analyses. The classification tests yield certain index properties of the soils important to the evaluation of soil behavior. In addition to the classification tests, direct and triaxial shear tests, and point load tests must also be conducted. The testing procedures of the tests are presented in the following paragraphs. All test procedures must be conducted in general accordance with the relevant ASTM standards, or any deviations must be mentioned.

Visual Classification

The soils are classified in accordance with the Unified Soil Classification System with certain other terminology, such as the relative density or consistency of the soil deposits, in general accordance with ASTM D 2487 and with engineering practice. In determining the soil type (i.e., gravel, sand, silt or clay) the term which best describes the major portion of the sample needs to be used The detailed descriptions of the samples examined in the laboratory are also usually presented in the geotechnical boring logs.

Moisture Contents

Natural moisture content determinations must be made on all samples of the fine-grained soil. The natural moisture content is defined as the ratio of the weight of water to dry weight of soil, expressed as a percentage. The results of the moisture content determinations should be presented on the logs of the borings, and should also be tabulated on the Summary of Geotechnical Laboratory Test Data.

Atterberg Limits

Atterberg limits must be determined on selected samples for the purpose of classifying soils into various groups for correlation. The results of the Atterberg limits tests, which include liquid and plastic limits, must be plotted on the Plasticity Charts and on the logs of the borings.

Grain-Size Analyses

Mechanical grain-size analyses (wet sieve) are to be conducted on selected soil samples to determine their grain-size distribution. In addition, hydrometer tests must be conducted on portions of the soil samples passing the No. 200 sieve. Mechanical grain-size analysis of bulk samples containing cobbles (material over 75 mm or 3-inch size) must be performed in accordance with ASTM D2487, Appendix X3, "Test Method for Classification of Soils for Engineering Purposes." The results of the mechanical grain-size analyses and hydrometer testing must be presented in the form of grain-size distribution curves on the Mechanical Analysis (Sieve Test) and Hydrometer Analysis charts. Since the USCS classification system is limited to the portion of the sample passing the 75 mm (3-inch) sieve, the material equal to or over 75 mm (3-inch) size must be removed in the laboratory prior to determination of the particle size characteristics. In order to obtain an accurate measurement of the entire range of particle sizes within the bulk samples, the total weight of the fraction retained on the 75 mm (3-inch) sieve is determined. The estimated maximum size of the largest clast within the plus 75 mm(3-inch) fraction is determined from measurements of whole clasts or radius of curvature for broken clasts. These maximum clast size measurements determine the end point of the cobbles on the grain-size curves.

Unit Weights

The unit weights, or densities, of undisturbed Shelby tube samples must be determined in the laboratory. First, the dimensions of selected cylindrical samples are measured, the samples weighed, and the wet unit weights calculated. Then, after oven drying, the moisture contents of the samples are determined and the dry unit weights are computed. The dry unit weights of the specimens must be tabulated in the Summary of Geotechnical Laboratory Data.

Blow Count and Relative Density of Cohesionless Soils

The relative density D_r of a granular material is defined as:

$$D_r = (e_{max} - e_{situ}) * 100 / (e_{max} - e_{min})$$

where e refers to void ratio. The relative density is commonly used as a measure of how tight the packing of grains is in a granular soil. This in turn is used to estimate the modulus, angle of internal friction, and the compaction density of the soil. There are correlation charts used for converting the blow count to relative density and to angle of internal friction for granular materials.

Blow Count and Unconfined Compressive Strength of Clays

Similar correlations are used by soil engineers to convert the blow count to unconfined compressive strength of fine grained soils. And can be used to determine the swelling capacity of fine grained soils. The activity of the clay can be determined using the definition:

Activity = Plastic Limit / % of clay fraction.

Although correlations between blow count and other parameters such as relative density and unconfined compressive strength are used in practice, it is highly recommended that a laboratory and field testing program be conducted to determine the same in order to obtain a better estimate of vital design inputs.

Field Shear Strength Tests

One of the most important engineering properties of soil is its shear strength. Shearing stresses in the backfill over an underground conduit, such as a sewer or culvert, exert a great influence upon the load to which the structure can be subjected to, in service. In fact there is hardly a problem in the field of geotechnical engineering, which does not involve the shear strength properties of the soil in some manner or the other. The in-situ shear strength of the undisturbed samples is estimated using a Torvane shear device and/or a pocket penetrometer. The Torvane is a hand-held vane apparatus. The vanes are inserted into an undisturbed soil specimen and a torque is applied to the vanes through a calibrated spring. When the soil fails in shear around the vanes, the apparent undrained shear strength of the soil is obtained from the spring calibration. The pocket penetrometer is a small hand-held probe consisting of a 6.35 mm (1/4-inch) diameter rod and a calibrated spring. The force necessary to push the rod into the undisturbed soil specimen is measured and correlated with the undrained shear strength of the soil. The unconfined compressive strength of the soil is approximately equal to twice its undrained shear strength. The estimated shear strength values must be presented on the logs of the borings, and must also be tabulated on the Summary of Geotechnical Laboratory Test Data.

Laboratory Shear Strength Tests

It is recommended that shear strength tests be conducted on undisturbed samples, in the laboratory to determine strength estimates better, in a more controlled environment.

Direct Shear Tests

Drained direct shear tests are performed on undisturbed samples of fine-grained soil obtained from a Shelby tube from a boring. A vertical (normal) load is applied and the sample is inundated with water. The specimen is permitted to consolidate under the applied normal load. After consolidation, the sample is then sheared laterally at a constant strain rate, which is determined by an evaluation of the observed rate of consolidation. For each sample tested, the maximum shear stress versus applied normal stress at failure is plotted. Also, shown on the plot is an interpreted failure envelope, which defines an apparent effective cohesion, c', (at zero normal stress) and an effective angle of shearing resistance, ϕ'. Values of effective cohesion, c', and angle of shearing resistance ϕ', derived from the testing are in kN/m^2 (psf) and degrees, respectively.

Triaxial Shear Tests

The maximum effective shear strength that a soil exhibits under a given set of loading conditions is defined as the peak shear strength. Isotropically consolidated undrained triaxial shear strength tests with pore pressure measurements (CIU tests) are performed to determine the effective shear strength parameters of selected soil samples. Back pressures are used to achieve both rapid and, for practical purposes, complete saturation of the test specimens. The results of the triaxial tests are summarized graphically on the Consolidated-Undrained Triaxial Compression Test (CIU) plots. The plots also show the Mohr rupture envelopes for the series of tests that are performed.

Soil-Pipe Interaction

Most pipe materials, except reinforced concrete and asbestos cement, are flexible or semi-rigid pipe materials which rely heavily on the surrounding bedding and backfill materials and native soil conditions to carry most of the applied load. Therefore, it is important to characterize the stiffness properties of surrounding imported and native soils accurately in the design of wall characteristics. Appropriate design criteria should incorporate sound principles of soil-pipe interaction. It is also equally important to recognize the assumptions involved in using various design equations given in the published standards and textbooks on this subject.

Modulus of Soil Reaction, E'

The modulus of soil reaction, E’, characterizes the stiffness of soil backfill placed at the sides of the buried pipelines. Deflection is an important criterion in the design of buried pipes. The most common formula used to calculate deflections for design purposes is the Modified Iowa formula, which includes the empirical modulus of soil reaction, E’, to account for the restraint provided by the soil backfill at the sides of the pipe. Because of its empirical nature and the wide variation of values back-calculated from field measurements, the parameter E’ can introduce a large degree of uncertainty in calculated deflections. This necessitates the procedure to estimate E’ as accurately as possible. Jeyapalan and Watkins (2004)’s work that is used worldwide in engineering practice have taken the following factors into account:

- Native soil type
- Native soil compaction density
- Modulus of native soil
- Trench material type
- Trench material compaction density
- Modulus of trench material
- Size of pipe
- Pipe stiffness-soil stiffness ratio
- Depth of cover
- Trench width-Pipe diameter ratio
- Location of the water table

Compaction Tests of Bedding and Backfill

Compaction of soils is a process by which the air in the voids of the soil at site is driven out with the aid of mechanical equipment. Some moisture is used to lubricate the soil particles in order to increase the efficiency of the energy transfer from the compaction equipment to the soil. The density, load-bearing capacity, modulus, and shear strength of the soil compacted are increased. Also, the permeability of the soil is decreased by compaction efforts. Thus, it is desirable to compact the soil around pipelines in accordance with proper specifications to reduce the compressibility of the backfill and the native soil. One of the objectives of the pipeline design engineer is to choose the most effective backfill material for the given project and to control compaction in the field in order to meet the design and installation specifications. Usually, the backfill material is tested in the laboratory either using the standard or modified proctor test. The compaction curve is established in the laboratory to determine the ability of the soil to compact at various moisture conditions. Once the specifications are chosen by the engineer responsible for the design of the pipeline, it is the engineer's responsibility to ensure that the contractor is meeting this specification for soil compaction adequately in the field. This is usually done by performing sand cone, balloon or oil methods or nuclear density test in the field along the length of the pipeline. It is important to recognize the details of the standard and modified proctor density tests in accordance with AASHTO T99 or T180. It is not right for the engineer of record to require both types of tests on pipeline projects from contractors due to the fact that by having results on one test, it is easy to obtain the results from the other test. The total energy imparted to the 5 layers in the modified proctor test performed in a geotechnical laboratory is much greater than that to the 3 layers in the standard proctor test. Because of this major difference, the compaction curve from a modified proctor test will tend to plot higher and to the left of the compaction curve from the standard proctor test on the same soil sample from a pipeline project. In summary, the relationship between relative density, standard proctor, and modified proctor density can be used to convert from any one of these three to the other two.

Geotechnical Design Summary Report

In summary, as a minimum the following geotechnical information will be required for the design of pipelines/tunnels:

- Standard penetration values;
- Soil unit weights;
- Particle size distribution;
- Shear strength;
- Atterberg limits (liquid and plastic limits);
- Moisture content;
- Ground water elevation, historical data, and seasonal fluctuations.

During the subsurface investigation and the soil boring program chemical and environmental analysis of soil and groundwater samples must be performed. In addition potential sources of pollution must be identified along the alignment. Any Geotechnical Design Summary Report that is prepared for the project should provide the following information:

- Project summary and Sources of information;
- Geologic setting including regional geology;Geologic features of design and construction significance such as engineering properties, bedrock weathering, geologic hazards, ground water, and surface hydrology;
- Human-made features of engineering, design, and construction significance such as existing structures, buildings, foundations, utilities, and contaminants;
- Anticipated ground behavior and construction problems including impacts of open-cut excavation for pipelines, tunneling/microtunneling methods and equipment selection, previous experience in similar soil conditions, effects of construction, cultural and environmental constraints, and instrumentation and monitoring requirements;

- Excavation method including methods considered and evaluated, methods not allowed, and rationale;
- Ground support system including primary support system and final lining;
- Design of ground support including assumptions and considerations, minimum support requirements, responsibility for design and safety, excavation and support sequence, loading conditions, operational requirements, basis of analysis, and ground water control;
- Matters that should be included in the construction specifications including reasons for important or unusual requirements, special conditions, and allocation of risks;
- Anticipated quantities at specific project features such as settlement, subsidence, heave, ground water inflow, occurrence of boulders and size.

The Geotechnical Design Summary Report usually forms part of the contract documents in most projects.

Conclusions

The following conclusions can be made:

1. Civil engineers are making misjudgments in designing, specifying, testing, approving, and commissioning new pipeline projects. These decisions are costing the owners and the public enormous amounts of money at a time when civil engineers are campaigning to have access to even more funds to repair and rehabilitate our aging pipelines.
2. This can be attributed to no formal training provided in any of our colleges for pipeline design and construction and the practice of pipe vendors providing most engineering input to the engineers of record, even though most of these sales representatives have no background in engineering.
3. It is great that we civil engineers are inventive enough to sell new words like condition assessment based asset management, infrastructure, total quality management, criticality factors, etc. to our trusting clients, but we also need to pay closer attention to sound principles of pipeline engineering and geotechnical engineering to eliminate mistakes.
4. Engineering firms should be requiring proper geotechnical baseline input and staffing the pipeline projects with senior engineers with appropriate experience.
5. This paper provides some guidelines on what to include in a quality geotechnical baseline investigation and how good interpretation of geotechnical data could offer a successful pipeline project.

Reference

Jeyapalan, J.K, and Watkins, R.K. “Modulus of Soil Reaction Values (E’) for Pipeline Design,” in ASCE Journal of Transportation Engineering, Vol. 130, No.1, January 1, 2004, 43-48.

HORIZONTAL DIRECTIONAL DRILLING INSTALLATION OF SEGMENTED PVC WATERMAIN PIPE IN RICHMOND, CANADA

James Young, P. Eng.[1] and Preston Creelman, P. Eng.[2]

ABSTRACT

The City of Richmond is part of the Greater Vancouver Regional District in British Columbia, Canada and is located on a flood plain with elevations ranging from approximately 0.6 metres to 2.5 metres above sea level. These physical attributes present numerous challenges and opportunities with underground utility design and construction. Design and construction of water mains in combination with soft soil, river delta ground conditions in particular lends itself to trenchless construction opportunities.

The City established an Asbestos Cement water main replacement program in the 1990's and the Shellmont watermain replacement project was designed to be undertaken using the conventional "open trench" procedure. The contractor proposed the use of HDD and segmented Cobra Lock PVC C900 DR18 pipe as an alternative, "trenchless" method as an innovative opportunity and one that would reduce costs and provide many tangible benefits to residents adjacent to the work. Implementation of this alternative method proved to be successful and was recognized as an opportunity for future water main installation work.

BACKGROUND

The City of Richmond is part of the Greater Vancouver Regional District in British Columbia, Canada (see Figure 1) and has a population of approximately 185,000 in a region of overall population of approximately 2.2 million. The City is located on a flood plain with elevations ranging from approximately 0.6 metres to 2.5 metres above sea level. These physical attributes present numerous challenges and opportunities with underground utility design and construction. Design and construction of water

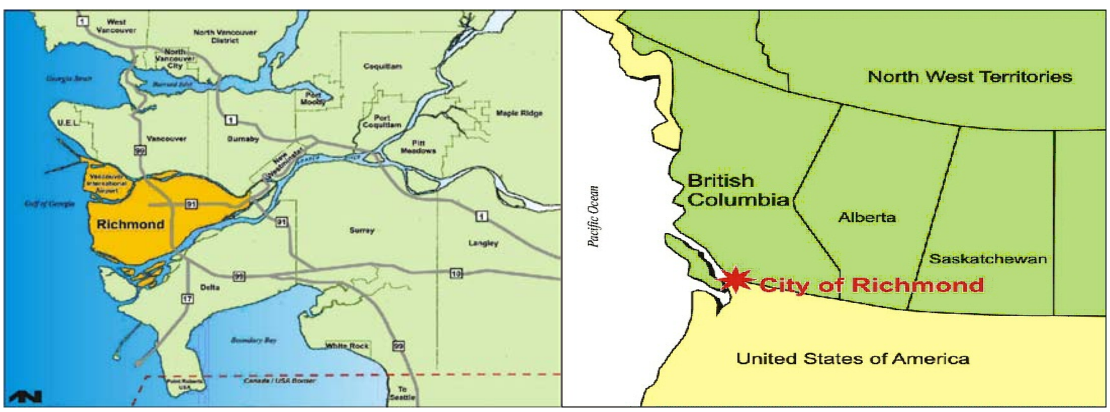

Figure 1 – City of Richmond, British Columbia, Canada

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mains in combination with soft soil, river delta ground conditions in particular lends itself to trenchless construction opportunities.

Horizontal directional drilling is gaining popularity in the Greater Vancouver area as an accepted method of installing watermains. In the City of Richmond, over three kilometers of aging 150mm diameter Asbestos Cement pipe was replaced using horizontal directional drilling and Cobra Lock™ PVC C900 DR18 Pressure Pipe, a first in Canada and possibly North America.

CONSIDERATION OF HORIZONTAL DIRECTIONAL DRILLING

The City established a proactive watermain replacement program in the 1990's with particular emphasis on replacing the asbestos cement (AC) mains. AC mains were installed as the small diameter watermain material of choice over the period 1952 to 1985 in the City and were in common use throughout the municipal sector in most areas of Canada and North America. In general, those AC mains that are under 300 mm in diameter have been prone to a decreased life expectancy through an accelerated loss of structural strength leading to diminished water system reliability. There is also the unfavorable public perception of drinking water being in contact with asbestos.

AC water mains in the City of Richmond have been found to be particularly vulnerable to the local conditions, specifically groundwater and the soil composition. As the City is located essentially at sea level, the phreatic surface is influenced by the tidal cycle and typically ranges from approximately 1.0 metre to 2.5 metres below ground level. During rainfall events it is not uncommon for the phreatic surface to be at the ground surface.

The City's AC pipelines are eroding from both the inside and outside of the pipe. Water supplied from the Greater Vancouver Water District has a low ph that accelerates leaching of cement mortar from the inside wall of the AC water pipes. Similarly, the high water table and aggressive soil in Richmond accelerates the cement mortar leaching from the outside pipe wall of the AC water mains. These two factors have combined to reduce the effective life of the City's AC pipelines below the anticipated 75 year design life.

Given the reduced life expectancy of AC watermains and the associated impacts on system reliability, the City has taken a proactive role in this regard by developing an AC watermain replacement program.

The Shellmont AC watermain replacement project was designed as shown on Figure 2 to facilitate conventional "open-cut trench" construction procedures through a residential area. Open-cut construction in residential subdivisions typically has considerable impact on residents and motorists which is normally managed through a proactive communication program and contract imposed restrictions with regard to the contractor's activities on site. Accordingly, the contractor proposed the use of horizontal directional drilling and segmented Cobra Lock™ pipe as an alternative, "trenchless" method as an innovative opportunity for the City's consideration.

The City had successfully completed directionally drilled watermain projects in the past using continuously "welded" pipe, usually High Density Polyethylene. While C900 PVC pipe has widely been used throughout the City on standard open-cut projects, the City had no previous experience with its use in a directionally drilled application. Furthermore, the City was unaware of any previous applications of this scope (approximately 3 km of water main) in Canada or North America.

The soil conditions in general in the City represent excellent conditions for trenchless technologies, including horizontal directional drilling. Figure 3 illustrates a typical soil profile whereby there is generally a thin layer of construction related granular materials at the surface overlying an approximately 2 to 4-metre thick layer of a soft to firm silt mixed with clay. It is within the silt/clay layer where main installation is completed. The City could proceed at this location and others with a great deal of confidence that uniform conditions would exist and that there would be no significant obstacles, i.e., boulders, etc., that would impede progress. Accordingly, the City completed a detailed review of this opportunity and considered the following main items.

Advantages

1. Minimized disruption to residents and traffic – Open-cut procedures on the Shellmont project (and most other watermain replacement projects in the City) would have considerable impact on residents in the vicinity of the work. Specifically, road and driveway access is hindered, noise levels are generally high, the area remains 'dirty' for extended periods of time and there is a general inconvenience to the neighborhood. Horizontal directional drilling would eliminate the majority of driveway and access and traffic conflicts and minimize disruption to the neighborhood in general. Installation of PVC pipe in particular through horizontal directional drilling could be completed one pipe at a time thereby eliminating the need to string long lengths of pipe that would certainly cause driveway and traffic disruption.

2. PVC pipe versus High Density Polyethylene pipe – From the City's viewpoint, both PVC and HDPE pipe represent excellent watermain building materials that are well suited to local conditions and facilitate horizontal directional drilling. As the City completes the vast majority of new and replacement watermain construction with PVC pipe, the operations and maintenance program has been geared in this direction accordingly. The use of HDPE pipe would require significant staff training and purchase of relatively expensive equipment. As the PVC pipe would be continuously restrained, seismic properties similar to High Density Polyethylene pipe were anticipated.

Figure 2 – Shellmont AC Watermain Replacement Project Typical Plan and Profile

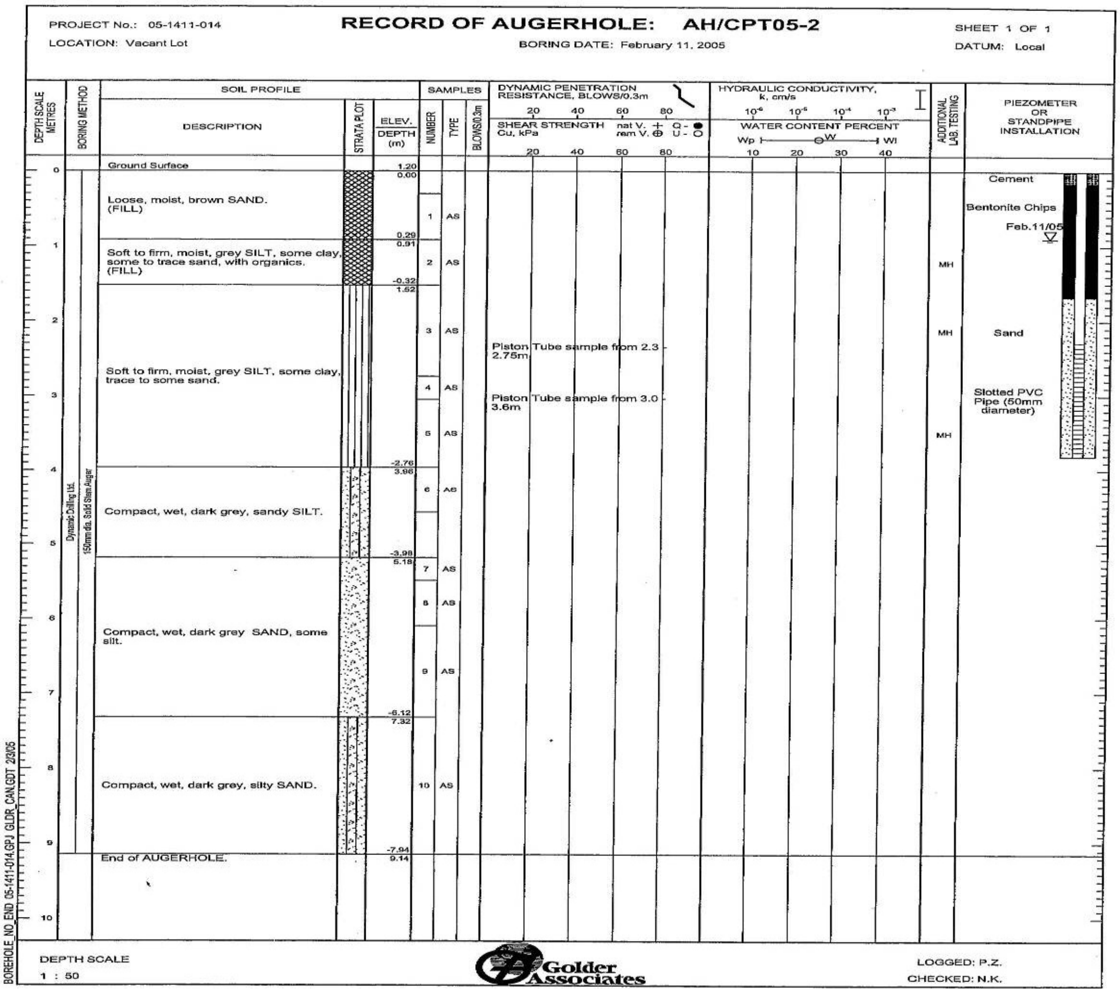


Figure 3 – Typical Soil and Groundwater Conditions in Richmond, British Columbia, Canada

3. Cost – As the contractor proposed the change to complete the work through the use of horizontal directional drilling as opposed to the tendered open-cut method, the City was in a position to request a credit to the contract.

4. Sustainable practice – An important measure of success to the City is to be recognized as leaders in sustainable practices. Installation of the watermain through directional drilling would facilitate this success descriptor as the need to import granular bedding materials and various surface restoration requirements including asphalt would be entirely eliminated and/or reduced.

5. Quicker method of installation – Disruption to residents was further minimized as the schedule provided by the contractor for watermain installation through horizontal directional drilling allowed for quicker completion as compared to the open cut method.

Disadvantages

1. Risk associated with a technique new to the City – The City had completed numerous horizontally directionally drilled projects in the past, but had no previous experience using PVC pipe over the distances contemplated on the Shellmont project. The use of Cobra Lock™ PVC C900 DR18 Pressure Pipe was also new to the City, but does comply with the City's standard specifications.

2. Unknown utility conflicts – Utility conflicts become obvious through open-cut installation methods as the excavation proceeds. The discovery of a direct conflict with an unknown utility(s) would require open cut excavation(s) thereby defeating the purpose and advantage of horizontal directional drilling. The City also had previous experience with other organizations whereby they completed their utility installation, i.e., gas main, through horizontal directional drilling, didn't realize there was a conflict, and installed their pipe through the City's existing sewer pipe.

3. Concerns with reduced PVC pipe structural capabilities

   a) The City typically completes PVC watermain construction through conventional open-cut methods and realizes the benefit of engineered backfill, compacted to a minimum of 95% Proctor density. As pipe installation through directional drilling would be completed in native ground, the City would not realize this benefit.

      The recommended maximum deflection for PVC pipe is generally considered to be 7.5%. Calculations completed on the construction basis condition (empty pipe with live load conditions) using the Modified Iowa Formula (below) with a Modulus of Soil Reaction value of 500 psi provided for an acceptable result of approximately 0.8% deflection which was considered to be a minor compromise.

$$\frac{\%\Delta Y}{D} = \frac{\left(D_L K P + K W'\right)(100)}{\left(0.149\dfrac{F}{\Delta Y} + 0.061E'\right)}$$

   b) The PVC pipe proposed required a smaller distance between the entry and receiving pits because of tensile strength limitations as compared to high density polyethylene pipe. The use of PVC pipe would require more excavations throughout the project along with the associated disruption to traffic and residents.

   c) While PVC pipe may be pulled onto a curved path, the pipe curvature limits the ability to tap the main to install services as part of the construction project and in future installations. Tapping a curved PVC pipe is discouraged by the City of Richmond. The portions of the watermain requiring a curved alignment would have to be complete through traditional open-cut methods.

The City reviewed the merits of the advantages and disadvantages and concluded that proceeding with directional drilling of segmented PVC pipe was warranted and would provide an overall benefit to the City.

CONSTRUCTION

Horizontal directional drilling is one of several trenchless construction methodologies whereby a drilling bit is guided through soil to create a round cavity, which will stay intact for at least several days. The drill head is propelled by adding segments of rod as the head proceeds forward; start-and-stop procedure. Once a cavity is created, the drill bit is removed and a pulling adaptor is attached to the drilling stem. A length of PVC pressure pipe is affixed to the adaptor. As the adaptor is pulled back to the rig, segments of drill rod are removed. Simultaneously, segments of Cobra Lock™ PVC Water Pipe are coupled to the adjacent pipe. This process is repeated until the adaptor returns to the rig and all of the watermain is in place.

Drilling - The drill rig used to install the pipe on this project was a DD30 – modified to 180 kN (40,000 lb) pull. Maximum rotary torque was 7,000 Nm (5,000 ft lbs). A 300-350mm (12 – 14 in) diameter wing cutter / back reamer was used to drill the hole. The line was installed in 90 to 120-metre runs, with an average of 10 to 14 hours to drill the cavity.

Spoon-style Bit suitable for use in wet clay

Pullback – Drill rod with reamer attached to swivel in front of lead pipe, ready to commence installation

Pullback - This was followed by a compaction reamer ahead on a swivel which was attached to the pipe to pull in the new Cobra Lock™ Segmented PVC C900 DR18 Water Pipe. No drilling mud was needed because the soil layer was a damp, firm silt mixed with clay (see Fig. 3). Actual pipe typically took 1 ½ to 2 hours. Pipe jointing was done quicker than the drill rod section removal, so pullback went smoothly. Maximum pull encountered over the course of the job was just over 22 kN (5,000 lbs). This was well below the limit of the rig and the maximum allowable for the Cobra Lock ™ Pipe (see Table 1).

Pipe Pull Force

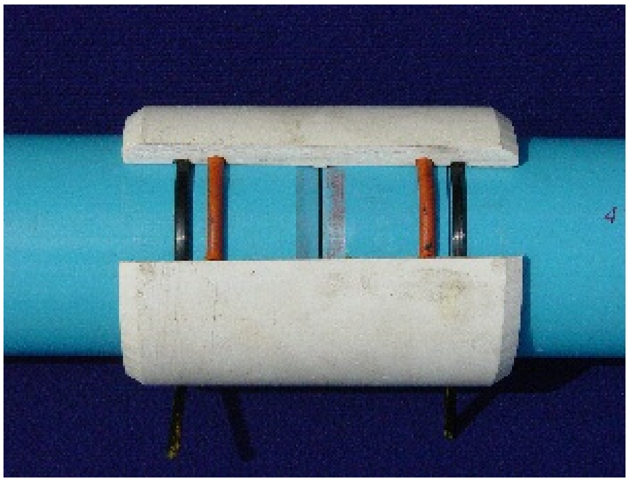
Maximum Pulling Forces as shown in the following table were determined from applying a safety factor of 2 to the yield strength of coupled joints tested in tension. (Note, the pipe wall itself can withstand much greater axial pull forces; 1.8 to 2.7 times the forces noted in the table for the system.)

	Royal Cobra Lock™ System	
Nominal Size mm (")	**Maximum Pulling Force** kN (lbs) **Class 150 (DR18)**	
	Tightest Bending	**Straight (no bending)**
100 (4)	29.8 (6,700)	36.4 (8,200)
150 (6)	40.0 (9,000)	56.9 (12,800)
200 (8)	80.0 (18,000)	112.0 (25,200)
250 (10)	113.8 (25,600)	156.4 (35,200)
300 (12)	117.3 (26,400)	182.7 (41,100)

From Royal Pipe's Cobra Lock System™ brochure dated 4/2004

Pipe/Joint Assembly

Cobra Lock Water Pipe is a PVC CIOD pressure pipe manufactured in 6.1 meter (20 ft) lengths to meet the requirements of AWWA C900. Each pipe is beveled and grooved at each end. Along with each length of pipe, a PVC coupling is provided. These couplings have two rubber o-rings placed in grooves which have been machined on the inside, near the centre of the coupling. Between the o-rings and the open end, there are corresponding grooves machined to align with the grooves at each end of the pipes, with an angled entry hole. When a spigot end is inserted into a coupling, a square nylon spline is pushed through the entry hole and around the cavity until it returns to the "entry hole" spot. The splines "lock" the pipe and couplings together, while the o-rings provide the pressure seal between the pipes. This mechanical jointing procedure allows for a large measure of flexibility during assembly of the pipeline.



Coupling – Cut-away of assembled coupling showing center stop, o-rings and splines.

CONCLUSION

Trenchless construction methods are being used more commonly in British Columbia, especially Horizontal Directional Drilling as described above. Residents gain the added benefits of less traffic disruption/congestion, reduced dust and site debris, minimal restoration of properties, and participation in a more sustainable community. All these attributes were realized on this City of Richmond project including reduced costs. These hidden/intangible advantages to the municipalities are recognized by industry as opportunities for future works.

References

1. Asbestos Cement (AC) Class 150 Watermain Piping Phase IX – 2006 Condition Evaluation Program Summary Report 1, Levelton Consultants Ltd., G.W. Stark, E.I.T. and R.S. Charlton, P. Eng., June 29, 2006.
2. Asbestos Cement Watermain Time to Failure, Lloyd Bie, P. Eng., January 20, 2005.

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DAVENPORT RANCH, AUSTIN, TEXAS
CROSSING LAKE AUSTIN USING
HORIZONTAL DIRECTIONAL DRILLING

Spenta Irani , PE[1], Paul Savard, PE[2], Mike Boyle, PE[3], and Phil Salyers[4]

Abstract

The Austin Country Club (ACC) and Davenport Ranch development needed to decommission its independent wastewater treatment facility and connect to the City of Austin's wastewater conveyance system. To support this initiative, the City of Austin developed a Master Plan to build a new Lift Station in the ACC area and discharge the wastewater to the city's Cross Town Tunnel. To implement their plan, the City would need to design and construct a 4,000 foot force main that crossed the ACC golf course and Lake Austin. Concurrently, the City partnered with Austin Energy, the local electrical supplier, to bundle the force mains with a series of electrical conduits, allowing Austin Energy to extend electrical service between the Davenport Ranch development and the Austin Energy transmission system on the other side of Lake Austin. This paper discusses the many challenges addressed during the design and construction phases of the project, ultimately resulting in a successful HDD installation, providing vital new wastewater and electric infrastructure to the residents and businesses surrounding Davenport Ranch.

Introduction and Project Overview

The City of Austin investigated a number of construction methods to install a new 12-inch wastewater force main across Lake Austin and Austin Country Club (golf course), concluding that horizontal directional drilling (HDD) was the most favorable method. HDD is a trenchless construction technique that uses guided drilling to create an arc profile. This technique is most applicable to long distance crossing beneath rivers, lagoons, landfalls, and sensitive or highly urbanized areas. The process involves three main stages: 1) Piloting (drilling of a pilot hole); 2) Reaming (pilot hole enlargement in stages; 3) Pullback (installation of pipe).

From a design perspective, using HDD techniques to install large diameter pipe is more complex than traditional open cut techniques. However, the decision to use HDD must take into account the social costs such as noise and disruption of open cut trenching, measured against the HDD's benefits of reduced environmental stress,

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improved margins of safety, and eliminating risk to existing facilities. The social and economic benefits clearly justify the additional design cost for this type of solution. However, installing force mains and the electrical lines using HDD techniques created several specific challenges that were resolved during the design and construction phases of the project. This paper discusses both design and construction aspects of the individual project elements concurrently.

The City considered a number of alternative alignments for the new utilities, evaluating a dual 2,000-foot segmental installation, as compared to a single 6,000-foot continuous installation. In the end, the final configuration installed two 2,000 foot long segments using horizontal directional drilling (HDD). Each segment consisted of two 12-inch diameter HDPE force mains (one acting as a redundant service) and six 3-inch diameter electrical conduits, completely encased inside of a single 30-inch steel casing.

***Figure 1** – Davenport Ranch HDD Site: The two combined Horizontal Directional Drilling (HDD) segments (shown in pink) exceed 4,000 feet of subsurface improvements, installed with no disruption to the golf course, Lake Austin, the public boat ramp, or to Loop 360 and the Pennybaker Bridge.*

Segment 1 parallels the southern shoreline of Lake Austin. The pipeline starts at the maintenance area located about 500 feet south of the Lake shoreline and centrally within the golf course property. Along this alignment, the ground elevation varies between 494 feet and 515 feet above mean sea level (msl).

Segment 2 follows a north/south orientation parallel to the Loop 360 Bridge on the eastern side of the highway crossing below Lake Austin. The elevation near the Loop 360 highway boat ramp is approximately 500 feet above msl. Elevation rises sharply on the north side of the Lake, where the highway was cut through a segment of limestone formation. The Lake is approximately 600 feet wide at this location. At the termination point along Loop 360, the elevation is approximately 602 feet above msl.

Segment 2 negotiated nearly 150 feet of elevation change between the entry and exit points passing through the limestone substrate. Entry point for this segment was located within a 50-foot by 70-foot area, just east of the Loop 360 bridge. The exit was approximately 2000 feet north, between the shoulder of Loop 360 and the cliff. Laydown area for this segment was extremely tight and consisted of a 15 foot wide strip of land between the shoulder of Loop 360 and the adjacent cliff.

Developing Project Consensus:

While constructing this project was technically feasible with an alignment that placed an open cut trench through the adjacent residential neighborhoods and supporting the force main on the bridge, the drastic impacts of "cut and cover" installation were expected to stir strong feelings of resentment and resistance to the project. Further, the City recognized that there are extensive environmental consequences when installing open cut force mains adjacent to a river bed. Using HDD technology for installation of these lines mitigated these concerns, making it feasible to eliminate the force main alignment through the very vocal residential area and avoiding that pocket of organized resistance for the project. The HDD solution met all the goals of the City and addressed their infrastructure needs without any "human cry" that would have resulted from any undue disturbance to the area.

The success of a previous 24-inch water main HDD crossing beneath Lake Austin in 2000 (also designed by Jacobs) served to increase project confidence for the City as well as local stakeholders. The primary innovation that evolved from the design team's approach to project development focused on the opportunity to combine the electrical crossing in the same casing as the wastewater force mains. Once this idea gained support, the designers maintained communication between the project team and the various City departments to keep the project moving.

Early Planning Identifies Opportunities to Meet Multiple Objectives

In 2001 the City initially designed a crossing that would support the force mains from the Pennybaker Bridge (Loop 360 Bridge). The Texas Department of Transportation (TxDOT) was quick to remind the City that this structure is "the most photographed structure" in Texas, expressing their concern for any solution that may affect aesthetics of that facility. This TxDOT concern was considered in the original open cut design, which proposed a new sidewalk over the existing sidewalk with the force main between the two slabs. This "major" reconstruction caused additional requirements to bring the entrances up to meet ADA standards not involved in the original bridge design. Thus this option quickly became impractical and cost prohibitive. The HDD solution eliminated all these issues, while providing a safe, environmentally sound solution. At the same time, Austin Energy needed redundancy for their electrical service to the Davenport development. Their initial design considered erecting monumental steel towers to span the Lake. The ensuing public concern for the unsightly appearance of towers and right-of-way needs would have been enormous obstacles to overcome. Construction of these utilities also needed to avoid interfering with Lake Austin navigation while minimizing obstruction to the public boat ramp located below the Loop 360 bridge.

__Figure 2__ – Lake Austin and Penny Baker Bridge: The City and Austin Energy partnered to install new force mains and electrical conduits to serve the Davenport Ranch development near the picturesque Lake Austin and Penny Baker Bridge.

By combining the two utilities within a common casing, the project saved literally millions of dollars associated with an aerial electrical crossing and avoided costly, time-consuming negotiation with various agencies and public groups that would be required for either crossing.

Cost Effective Sharing by Compatible Utilities Offers Significant Advantages

Once HDD was selected, the City decided to take advantage of the economies of scale and add a redundant force main into the project. In this way, they could be assured of ample capacity and redundancy should one line ever need to be taken out of service. The City also recognized the potential for bundling the electric conduits within the remaining space within the casing. Once Austin Energy was approached, the solution to both utility's problems for serving Davenport Ranch became obvious.

Although constructing a force main across Lake Austin was unusually well-suited for the HDD method, the City was able to leverage this opportunity to provide cost sharing benefits by teaming with Austin Energy. This project combined multiple users for the crossing, offering fully independent benefits, while substantially reducing the overall cost associated with each entity. Austin Energy entered into a cost sharing agreement with City of Austin's water utility, thereby sharing the $5M construction cost while literally saving millions of dollars associated with spanning Lake Austin with separate aerial tower crossing for the electrical lines.

Innovative Project Packaging

The initial design consisted of the lift station, force mains and the gravity lines as a single construction package. While this package met the intent of holding one contractor responsible for construction, it severely limited the bidding and bonding capabilities of smaller contractors. Also, many HDD contractors did not bid the job (As Prime) due to the lift station component. This resulted in one bid which was cost prohibitive. In response, the team worked aggressively to implement an innovative construction packaging scheme which separated out the lift station, HDD components and open cut gravity line into three packages. Subdivision of this construction provided more opportunities to multiple contractors and resulted in an overall cost reduction of $2M for the complete project. Mears HDD, LLC was the successful contractor for the HDD project (Package 1) with a low bid of $5.2 M. The costs were well within what the City considers "acceptable tolerances", especially in light of the drastic rise in material costs (most notably steel and fuel), during the development of this program.

Managing Pipe Lay-Down and Work Area Constraints

An area west of a triangular shaped plot of land approximately 100 feet by 200 feet along its longest legs was used for beginning the HDD entry for Segment 1. The land surrounding the maintenance area is the open space of the golf course. The force main traversed beneath the golf course and a waterway inlet protruding into the ACC property from Lake Austin, just east of the Highway Loop 360 bridge, and terminate at an existing boat ramp below the Loop 360 bridge. Travis County maintains the boat ramp, while TxDOT owns the Right of Way. The paved area of the boat ramp is roughly 300 feet by 200 feet and is used year-round for launching recreational watercraft.

For HDD installation, a pipe lay-down area that is equal in length to the total installation is most desirable. Stopping during pullback to connect additional pipe greatly increases the risk of pipe becoming stuck in the hole. The need to identify suitable pipe lay-down areas for this project were complicated by the environmentally sensitive Lake Austin shoreline, the heavily traveled Loop 360, and the physical constraints posed by the cliff. Along Segment 1, the vegetated shoreline along Lake Austin, west of the boat ramp provided a good lay-down area. However, the active boat ramp needed to remain open to the public throughout construction. Therefore, the lay-down and pipe assembly area was short by about 200 feet due to difficulties in getting construction easement and a drainage channel separating the boat ramp and the lay-down area. Design and specifications made allowances for the pipe to be assembled in two sections and the final sections were welded during pullback operations.

Figure 3 *- The lay-down area for Segment 1 extended for 2000 feet between the Lake Shore and the vegetated land. An open, grassy area west of the boat ramp extends approximately 1,000 feet along the shoreline of Lake Austin; the land west of this strip is an overgrown vegetated strip consisting of large trees. Due to vegetation and other constraints the width of the lay-down area was limited to 50 feet.*

Lay-down area for Segment 2 was extremely tight and consisted of a 15 foot wide strip of land between the shoulder of Loop 360 and the adjacent cliff. Concurrent assembly of the Steel Pipe and HDPE pipe was not feasible in this tight area. Hence, the decision was made during construction to pull the steel pipe, followed with the assembly and pull of the HDPE conduits.

Understanding Subsurface Conditions

Preliminary Segment 1 geotechnical data indicated a shallow limestone formation (the Glenrose formation). However, after the initial borings were obtained, the data suggested the top of limestone dropped more than anticipated as the alignment approached the shore of Lake Austin. While the HDD method can negotiate varying amounts of soil and rock interfaces, the drill equipment operates optimally when traveling through a consistent, rather than a mixed, subsurface material. Negotiating undulating or varying soil and rock will wear out HDD drill equipment prematurely.

To manage this risk, a second round of borings located close to the alignment was obtained. These boring were located so that they would not provide a pathway for drill mud to break out during construction. The second round of data verified a more variable subsurface condition along the alignment that would consist of fill close to the surface, with an undulating layer of alluvial soils above a limestone formation. Fill at the golf course consisted of poorly compacted silt, clayey silt, clayey sandy silt and clayey silty sand with small to large limestone gravel, cobbles and boulders. Alluvium varied over short distances both vertically and horizontally. The soil/rock interface was found to be highly irregular with the potential to contain gravel, boulders and cobbles. The rock layer consisted of moderately dolomitized fine-grained limestone with a combined quartz-plagioclase content of less than 5 percent. With this data, our team was able to optimize the force main profile to operate within the hydraulic parameters of a lift station that was already in the final stages of design, while providing the HDD contractor information to properly prepare their bid.

A Geotechnical Baseline Report can be an effective risk management tool for HDD

The City elected to prepare a geotechnical baseline report to provide bidders a set of baseline conditions that would serve as a common basis for bid development. The GBR was intended to facilitate more accurate bidding without inflating the risks that may not eventuate. The GBR conveyed those baseline values that determined the anticipated conditions to be encountered by the HDD contractor during the course of the installation. The values provided common ground between the City, the City's consultant, and the City's contractor to evaluate conditions encountered and to balance the risks. The GBR provided a list of baseline values of rock and soil. The baseline conditions provided to the contractor were set on the density and compressive strength of the soil and rock. The anticipated construction impacts based on the baseline conditions set were presented in the GBR. From this, the City and contractor were to anticipate soil and rock that are susceptible to frac-out of drilling fluid and that drilling navigation would be negatively affected due to changing soil and rock characteristics.

By basing their bids on uniform information, the contractors were able to manage contingency bid amounts and the City was able to control its exposure to potential claims should subsurface conditions achieve the baseline conditions presented. For example, during construction of Segment 1, the undulating strata of the rock and soil was creating problems as the drill stem was slipping at the top of the rock and was hindered from entering the rock bed during the initial phase of pilot hole. Frac-out of drill mud occurred within 50 to 150 feet of entry location because the existing ground surface sloped at the approximate angle of the drill path resulting in a thin soil layer insufficient to hold down the mud pressures. This problem was addressed by a steeper entry angle drill angle along with a casing pipe. The steeper angle facilitated the entry of the drill bit into the rock, while the casing pipe maintained the recirculation of the drilling fluid at the entry point and eliminated frac-outs. The contractor did not claim a changed condition in part because this potential was identified in the GBR. The contractor was able to implement their contingency plans and mitigate for the frac-out conditions reasonably.

Entry/Exit Elevation Changes

The lake crossing had a 150-foot elevation change creating technically challenging entry and exit points. Our team evaluated all the pull-back stresses on the steel casing and the HDPE pipe taking into consideration the elevation, distances and the geological strata. Design of pipe pullback took into account the bridge height and the entry angle of the pipe, as all pipe had to be pulled below the bridge without damaging the beams.

Bundled Electrical Conduit Require Specialized Grouting

The capabilities of HDD are being challenged and pushed with every new project – challenges that recently would have been considered beyond the technical envelope are now routinely achieved. A combination of factors pushed this project to the edge of HDD's technical limits: the project's length, large diameter, but most importantly, the need for multiple conduits to accommodate the dual-purpose electric service and wastewater conveyance contained inside of a single casing pipe. To address the dispersion of heat generated from the electrical lines, the annular space within the steel casing had to be filled using a grout with a very low thermal resistivity of less than 90^{o} C-cm/W. The grout also had to be designed to travel long distances within limited intestinal spaces. To meet this design restriction, a grout mix with a time of Efflux of 25 seconds or less was designed. Further the grout had to maintain the time of efflux for a duration of more than 4 hours to facilitate pumping. To meet these requirements, extensive testing and investigation was completed during the design phase to develop a grout mix that could meet the thermal and viscosity characteristics, to facilitate grouting from both ends of the HDD casing.

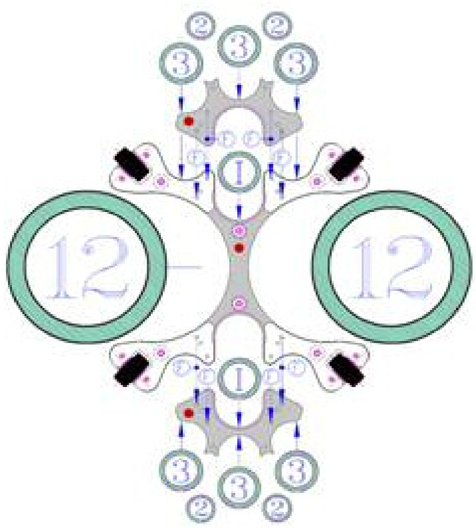
Our discussions with leading grout contractors found no applications where grouting was successfully completed over these distances unless the thermal resistivity limits were relaxed well over that required for our project. We helped the City resolve this challenge by hiring Geotherm (a specialized grout company) to develop a grout that satisfied each of these unique requirements. Our team prepared and experimented with several trial mixes consisting of locally available materials to determine a mix that met the project requirements. The key element of the mix that made this application a success was to use silica flour instead of sand along with other more traditional components of cement, fly ash and polymers to meet all requirements. This project proved that mixes reliant upon traditional sand in lieu of silica flour could not achieve the high pumpability requirements. Further, we realized that mixes with even very small quantities of bentonite (less than one percent), a technique commonly used to improve pumpability, causes significant water to be entrained, and would raise thermal resistivity well beyond our acceptable limit. Also, mixes with fly ash met all the requirements, but set relatively fast and could not maintain the efflux over the longer time durations. In the final design the fly ash was substituted by cement and plasticizers.

The final grout solution has a tremendous benefit to the engineering profession. Prior to developing the new grout design, projects of this complexity were either restricted to less than 1,000 feet or the electric service was de-rated which has costly long-term impacts because of the reduced load-carrying capacity of the electrical line. The combination of this grout mix and HDD expands the technical envelope, vastly increasing the number of HDD applications that can now be considered viable to maximize the efficiency of transmitting electric power. It also opens opportunities for more cost sharing between individual utilities, while reducing overall costs by allowing smaller casings to be used, and providing a tremendous risk reduction impact on ever longer HDD projects. By developing an improved grout mix to facilitate these twin 2,000-foot crossings in Austin, our data and research indicates the grout's flowable characteristics will enable project delivery in the range of 4,000 feet.

Specialized Casing Spacers Assist Bundling and Grouting

Our staff coordinated with a leading casing spacer manufacturer for the casing spacers that would be needed.

Figure 4 *- During construction, the contractor was able to modify the casing spacer design further to better assist with field assembly, to resist frictional forces during pullback, and to prevent damage to the HDPE lines from a cork-screw effects during the pull-back through the steel casing. Also, modifications were necessary to add two additional grout lines.*



A full size mock-up of the casing spacer system and carrier pipes provides useful opportunity to adjust grouting procedures before full-scale grouting begins

The contract documents required the contractor to assemble a mock set-up of HDPE bundles, along with casing spaces and grout tubes. This allowed field testing and evaluation of grout designs, procedures, and lab testing before the full-scale grouting operation begins. The mock up (shown in the figure) provided an opportunity to monitor grouting procedures and to identify improvements that otherwise could not be planned in the midst of the full-scale grout operation when every minute is critical toward the success of the grout operation. Upon request the grout mix developed during design was provided to the contractor to develop the field grout mix. A complete mock field test for a 50 foot pipe segment to evaluate feasibility of the casing spacers and grout operation, prior to installation was completed. This test also determined the approximate amount of grout that would be needed for the line segments. Minor changes to the design of spacers were completed to add additional grout tubes and provide back-up tubes if any of the smaller pipes clogged. Also, HDPE pipes were filled with water to avoid any risk of deformation as a result of heat of hydration during setting of grout. After successful mock-up tests the HDPE force mains, electrical conduits and grout tubes were welded and laid out for both the segments.

__Figure 5__ - Based on the mock up results, grouting operations were done in phases. The first two phases consisted of sealing the center sections of each line segment to create a plug. This was followed by grouting at either ends to completely fill the voids. Batch tests of the grout was collected at each phase to determine the Thermal Resistivity of the mixes and verified that grout batches met the specified requirements. Pressure and mandrel testing of pipes were completed after the grout was cured and verified the internal diameter of all conduits met the deflection limits specified.

Allowing for Changing HDPE Pipe Characteristics During Assembly

HDPE pipe exposed to direct sunlight had very high expansion. These pipes were contracting during the night hours and created loose bundles. Any minor pipe movement during the HDPE pullback operation had to be avoided. It was critical that pipe bundles and casings do not cork-screw and get stuck inside the 30-inch steel pipe.

__Figure 6__ - Bundling our pipes was done in the early morning hours and pipe was pulled inside the casing as the bundles were assembled. A special rack was fabricated to facilitate this operation. Additional tightness was obtained by utilizing a canvas strap to bundle the pipes prior to tightening the steel straps.

__Figure 7__ - A special rack was fabricated to facilitate the pullback operation while maintaining correct separation between the multiple HDPE conduits.

Flexibility Allows for Contractor Input to Affect Means and Methods

After evaluating the project and the layout space Mears started the set up of the entry pits and the exit areas. A drilling plan was submitted for approval which identified the required labor and equipment.

__Figure 8__ - A DD500 (CMS Rig) and DD140 (American Auger Rig) were used for the operations. 2 MCS-750 mud mixing/cleaning systems along with a combination of mud pumps were used for the drilling operations. A ParaTrack2 MGS Guidance System was utilized along with other equipment consisting of back hoes and trucks. The drilling fluid primarily consisted of fresh water and bentonite, with polymers to optimize performance.

Due to seasonal closing restrictions imposed by golf course, the Segment 1 installation was drilled first. The sequence of construction operations was as follows:

- Set up drill equipment and mud pit within the designated golf course area. Drilling, reaming and pullback of 30-inch steel pipe along Segment 1. Welding and testing operations along the layout space were done concurrently along with the drilling.
- Move drill ring to the boat ramp area and drill north to the exit point for Segment 2. Drilling, reaming and pullback of 30-inch steel pipe along Segment 2. Welding and testing operations along the layout space were done concurrently along with the drilling.
- Assemble and test HDPE bundles for Segment 1 and Segment 2. Complete mock grout test.
- Pull HDPE bundles for Segment 2, followed by Segment 1. Pressure test HDPE bundles and electrical conduits.
- Complete grouting for Segment 2 and Segment 1. A detail grouting and testing sequence was developed to plug the bottom portions of the segments and avoid any major issues during the grouting process.
- Final tie-in, including connections to MH, installation of electrical MH, Air Release MH's and testing was completed.

Construction Scheduling and Sequencing are Vital Management Tools

A detailed schedule was developed with the design and included in the contract documents as there were time restrictions for construction within the golf course. Also, location of the drilling equipment and layout space was pre-determined. The contractor had the option to install the casing along with the HDPE pipe and pull the bundle at one time, or pull the steel casing and then pull the HDPE conduits inside the casing.

The design of the force mains was completed and the project was advertised in July 2005. Bids were opened in September 2005 and the low bidder was Mears HDD, LLC for about $5.2M. Construction of the project started in January 2006 and was substantially completed in one year. All of the City's project management scheduling was maintained – even in light of the multiple bids this project incorporated to maintain adherence to the City's budget and time schedule.

Conclusions

The Davenport Ranch forcemain project applied innovative planning and design techniques to successfully control construction costs and manage risks on a challenging HDD project. The project installed wastewater and electric services in a single installation that greatly reduced major impacts to Austin Country Club golf course, Lake Austin, Penny Baker Bridge and Loop 360. A geotechnical baseline report provided flexibility for the HDD contractor to manage its means and methods to avoid construction surprises and delays while completing the project within time and budget. Special considerations for casing spacer and grouting techniques set during the design phase were tested before installation with a full sized mock up. The mock-up allowed improving grouting procedures before full scale installation was undertaken. The special thermal grout design achieved a mix with unusually low thermal resistivity characteristics, but that also could be pumped over long distances within small spaces. These unique aspects of the project help demonstrate how HDD applications can be applied to ever more challenging conditions and continue to emerge as a cost-effective technique to install subsurface infrastructure beneath bays, historical districts, sensitive structures, under functioning transit or transportation arteries – anyplace where disruption of existing facilities is prohibitive to advancing a project.

Simplified Methodology for Selecting Polyethylene Pipe for Mini (or Midi) – HDD Applications

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Abstract

A simplified methodology is presented to estimate the required pull loads on polyethylene pipe installed by a mini (or midi) - HDD process. A simplified procedure is also presented to evaluate the potential collapse tendency of the PE pipe during the installation or post-installation phases. The methodology and associated formulae are based upon approximations to the more complex set of equations and procedures provided in ASTM F 1962, *Standard Guide for Use of Maxi-Horizontal Directional Drilling for Placement of Polyethylene Pipe or Conduit Under Obstacles, Including River Crossings*. The objective is to provide a convenient means of identifying potentially problematic mini-HDD installations and/or to aid in the pipe (DR) selection process, in contrast to the extensive planning or analytical investigations characteristic of typical maxi-HDD projects. The proposed mathematical model reflects the major route parameters (bore length, planned bends) and buoyant weight for an empty PE pipe, and also accounts for unplanned curvatures (undulations) resulting from path corrections in a typical mini-HDD installation.

1. Introduction

Horizontal directional drilling (HDD) is a technique for installing pipes or utility lines below ground using a surface-mounted drill rig that launches and places a drill string at a shallow angle to the surface and has tracking and steering capabilities. In particular, mini-horizontal directional drilling (mini-HDD) is a class of HDD, sometimes referred to as "guided boring", for boring holes several hundred feet in length and placing pipes up to 12 in. diameter, at depths up to approximately 15 ft. Mini-HDD is appropriate for placing local distribution lines (including service lines or laterals) beneath local streets, private property, and along right-of-ways. In comparison, maxi-horizontal directional drilling (maxi-HDD) is a relatively sophisticated class of HDD, employed for boring holes several thousand feet in length and placing pipes up to 48 in. diameter, at depths up to 200 ft. Maxi-HDD is appropriate for placing pipes under large rivers or other broad crossings. Applications that are intermediate to the mini-HDD or maxi-HDD categories may utilize appropriate "midi" equipment of intermediate size and capabilities.

ASTM F 1962, *Standard Guide for Use of Maxi-Horizontal Directional Drilling for Placement of Polyethylene Pipe or Conduit Under Obstacles, Including River Crossings*, was approved in 1999, following development within the F17.67 Trenchless Technology Subcommittee of the ASTM. The ASTM document provides overall guidelines for a major horizontal directional drilling (i.e., maxi-HDD) operation, addressing preliminary site investigation, safety and environmental considerations, regulations and damage prevention, bore path layout and design, implementation, and inspection and site cleanup. One of the significant contributions of ASTM F 1962 is the provision of a rational, analytical method for selecting the polyethylene pipe strength based upon the estimated installation and post-installation (operational) loads on the polyethylene (PE) pipe (Petroff, 2006). Thus, ASTM F 1962 provides a means of determining project feasibility, as well as initial design information. Such results would be further refined by competent engineering expertise, including an analysis of pipe and soil characteristics and interaction, often including the use of relatively sophisticated software tools, such as *DrillPath* (Maurer, 1999).

Although considered convenient and practical to apply by experienced engineers for a maxi-HDD operation, the equations and procedures provided in ASTM F 1962 represent relatively complicated formulae, and a tedious methodology, when considering smaller, lower cost operations associated with typical mini-HDD applications. In addition, for most mini-HDD applications, such detailed analyses are not necessary or warranted since the installations are of relatively short distance and shallow depth, and typically well within the capability of the equipment and the product being installed (e.g., high density polyethylene pipe). Mini-HDD operations are often performed during an upgrade of a large community, comprising many individual installations, with any single installation not requiring or receiving extensive analysis. For isolated cases in which problems may arise, the low investment in the mini-HDD process allows a subsequent re-attempt, with appropriate adjustment, at a minimal cost penalty.

Nonetheless, some mini-HDD installations may be considered to be relatively critical, or approach limits with respect to the capability of the available drill rig and/or the strength of the product pipe being installed. It would therefore be valuable to have a practical, readily applicable methodology for judging the likelihood of success of the mini-HDD operation, as a function of the relevant parameters (route and pipe characteristics). The procedure would also provide a quantitative understanding of the relative importance of the fundamental parameters. The original principles and theoretically derived formulas that were provided in ASTM F 1962 for maxi-HDD installations have therefore been simplified and adopted to mini-HDD applications. With the recognition that such results necessarily provide only a rough, albeit generally conservative, estimate of that which is a very complex process, which is difficult to quantify even with the use of sophisticated techniques, the present method represents an extremely useful tool.

Although the methodology is primarily conveniently described with respect to mini-HDD installations, the results are also applicable to midi-HDD operations. The procedure is applicable to either high density polyethylene (HDPE) or medium density polyethylene (MDPE) pipe, both of which have characteristically low bending stiffness. It is anticipated that future efforts will further extend the methodology to pipes with greater stiffness, such as PVC, and possibly steel or iron.

The present results have been incorporated into the HDD training courses provided by the Center for Underground Infrastructure Research and Education (CUIRE).

2. Description of ASTM F 1962

Figure 1 illustrates a typical geometry for a major (maxi-HDD) operation, corresponding to a river crossing. The indicated path corresponds to that shown in ASTM F 1962 and comprises four segments, including those spanning the pipe entry to exit point (L_2, L_3, L_4) and the excess length (L_1) remaining after the span has been accomplished. Thus, the length of the actual crossing, L_{bore} , is given by

$$L_{bore} = L_2 + L_3 + L_4$$

In comparison, the total length of the product pipe, L, is slightly greater than the distance given by

$$L = L_{bore} + L_1$$

due to the additional length along the curved path. In some cases, the intermediate horizontal segment, L_3 , may be of zero length. Due to the typically low pipe entry angle, α, and exit angle, β, and gradual path curvature, the depth of the crossing, H, is small compared to the transition distances L_2 and L_4 .

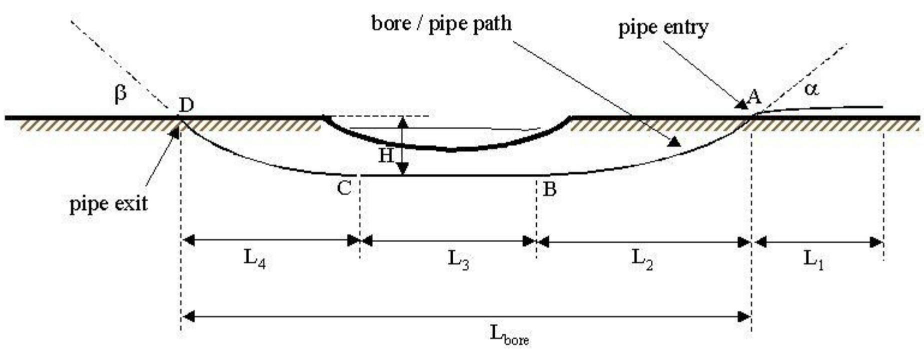

Figure 1. Nominal maxi-HDD Route (river crossing)

2.1 Pull force

Using the above terminology, ASTM F 1962 provides a set of recursive relations to predict the required pull force -- T_A , T_B , T_C , and T_D -- corresponding to the leading end of the pipe reaching point A, B, C and D (Figure 1). Thus,

$$T_A = e^{\upsilon_a \alpha} \cdot \upsilon_a \cdot w_a \cdot (L_1 + L_2 + L_3 + L_4) \quad (1a)$$

$$T_B = e^{\upsilon_b \alpha} \cdot (T_A + \upsilon_b \cdot |w_b| \cdot L_2 + w_b \cdot H - \upsilon_a \cdot w_a \cdot L_2 \cdot e^{\upsilon_a \alpha}) \quad (1b)$$

$$T_C = T_B + \upsilon_b \cdot |w_b| \cdot L_3 - e^{\upsilon_b \alpha} \cdot (\upsilon_a \cdot w_a \cdot L_3 \cdot e^{\upsilon_a \alpha}) \quad (1c)$$

$$T_D = e^{\upsilon_b \beta} \cdot (T_C + \upsilon_b \cdot |w_b| \cdot L_4 - w_b \cdot H - e^{\upsilon_b \alpha} \cdot [\upsilon_a \cdot w_a \cdot L_4 \cdot e^{\upsilon_a \alpha}]) \quad (1d)$$

where w_a and w_b are the empty (above ground) and buoyant weights of the pipe, respectively, and υ_a and υ_b. are the corresponding coefficients of friction. Equations 1 are sufficiently general to consider the possible implementation of anti-buoyant measures to reduce the otherwise high values of w_b for plastic (i.e., PE) pipe. In the absence of such anti-buoyancy measures, the maximum pull force will tend to occur towards the end of the installation; e.g., T_C or T_D .

Equations 1 are based upon conventional Coulomb friction, which assumes that drag forces on the pipe are proportional to the local normal bearing forces applied at the pipe surface, with the proportionality constant designated as the "coefficient of friction". Such bearing forces may be due to the dead (empty) weight of the pipe where above ground, the buoyant weight of the submerged pipe (possibly mitigated by anti-buoyancy measures), bearing/bending forces associated with pulling a stiff pipe around a curve, or bearing forces resulting from (previously induced) axial tension tending to pull the pipe snugly against any locally curved surfaces.

For the case of PE pipe, of typically low bending stiffness relative to that of the steel drill rods that created the gradually curved original bore hole path, the corresponding bearing/bending forces may be ignored. However, the tension-induced bearing forces are primarily dependent upon the cumulative bend angles, which may be significant, independent of the gradual nature or variable direction of such curves or degree of pipe bending stiffness, and are included in the analysis. Such effects compound, and in some situations may become the dominant source of drag, essentially controlling practical placement distances. This phenomenon is referred to as the "capstan effect" (i.e., the operating principle of the "capstan winch") and is the basis of the exponential terms in Equation 1. In particular, the following relationship illustrates the basic phenomenon for the idealized case of a weightless, flexible pipe:

$$F_2 = F_1 \bullet e^{\upsilon\,\theta} \tag{2}$$

where F_1 represents axial tension at the entry point of a bend of magnitude θ (radians), υ is the local coefficient of friction between the product pipe and bore hole wall surface, and F_2 is the required axial tension at the exit point of the bend. In practice, the impact of the actual weight of the pipe may be reflected in the preceding tension, F_1 .

2.2 Pipe collapse (buckling)

ASTM F 1962 also evaluates the possibility that the product pipe may collapse during the installation phase or during the post-installation (e.g., operational) stage. In general, the critical (buckling) pressure, P_{cr} , is given by

$$P_{cr} = 2\,E \bullet f_o \bullet f_R / \{(1 - \mu^2) \bullet (DR - 1)^3\} \tag{3}$$

where E is the material modulus of elasticity, μ the Poisson's ratio, f_o the ovality compensation (reduction) factor, and f_R the tensile stress reduction factor. The dimension ratio, DR, refers to the ratio of the pipe outer diameter to its (minimum) wall thickness. In a discussion of ASTM F 1962, Petroff (2006) explains the significance of these terms. The material properties, E and μ, for the viscoelastic HDPE pipe depend upon the load duration, f_o accounts for initial or subsequent out-of-roundness, and f_R recognizes a potential reduction in collapse strength in the presence of significant tensile loads during the installation phase. Petroff (2006) also provides an explanation of the possible sources and nature of the pressure loads on the pipe, including that due to hydrostatic pressure associated with drilling fluid or groundwater pressure, and asymmetric earth pressure that causes ring deformation, as well as the implications of their time dependent characteristics. In general, the detailed consideration of the interaction of the various phenomena, and the consequences for the product pipe, is relatively complex.

3. Simplification for Mini-HDD Applications

The detailed application of the set of Equations 1 to determine the required tensile load on the pipe during the installation phase, and Equation 3 to evaluate the possibility of pipe collapse during installation or the post-installation phase, would be tedious and represent overkill for typical mini-HDD applications. On the other hand, application to potentially problematic installations would be desirable, although not feasible, for most mini-HDD personnel, in spite of their being otherwise well-trained in this technology. Thus, the reduction of these equations to relatively simple formulae and practical procedures, albeit at a possible loss of precision, would be beneficial.

3.1 Pull force

In order to reduce the complexity of the set of Equations 1 for mini-HDD installations, the procedure is limited to PE pipe without the use of anti-buoyancy techniques. Substituting specific (conservative) values for several of the parameters, and a comparison of the typical magnitudes of the resulting calculations, allows a major simplification of the predicted pull force at the end of the installation, T_D. In particular, values of the frictional coefficients υ_a and υ_b are assumed to be equal to 0.5 and 0.3, respectively, and pipe entry and exit angles, α and β, are assumed to be 20°. Thus, it may be shown that Equation 1d can be simplified to

$$T_D \approx L_{bore} \bullet w_b \bullet (1/3) \qquad (4)$$

It is recognized that, under appropriate conditions and actual installations, it is possible that the pull force may achieve its maximum level prior to point D. However, with the present basic theoretical model, under the assumed conditions and conservative parametric values, the predicted tension at point D would be a maximum, or reasonably close in magnitude to a previously occurring (predicted) maximum value.

For mini-HDD installations, the above estimate T_D must be modified to account for the possibility of additional path curvature due to deliberate route bends as well as the likelihood of unplanned undulations resulting from path corrections. The presence of such characteristics in the final ("as-built") path will increase the required pull force, consistent with the capstan effect described above. These effects may be conservatively estimated by the applying the exponential term in Equation 2 to the tension T_D , such that

$$T_D{}^1 = T_D \bullet e^{\upsilon_b \theta} \tag{5}$$

represents the net final tension, for which the angle θ is selected as equal to the total additional route curvature. The latter may be expressed as

$$\theta = n \bullet (\pi/2) \tag{6}$$

where n is equal to the number of additional 90° route bends due to the cumulative route curvature, as described below. Considering the assumed value of υ_b of 0.3, combining Equations 4 - 6 then yields

$$T_D{}^1 \approx [L_{bore} \cdot w_b \cdot (1/3)] \cdot (1.6)^n \tag{7}$$

It is interesting to note that Equation 7 does not include the depth of the route, H, which is explicitly included in the original Equations 1. The mathematical simplifications have essentially eliminated this dependency as a relatively minor effect, as appropriate for the present purposes, and will be verified to be unnecessary to adequately estimate the pull load (see Section 5).

Additional path curvature. The value of n in Equations 6 and 7 may be expressed as

$$n = n_1 + n_2 \tag{8}$$

where n_1 is the effective number of deliberate/planned 90° route bends, and n_2 is the cumulative curvature due to the unplanned undulations. For example, if a deliberate horizontal (planar) bend of 45° to the right, in order to avoid an obstacle or follow a utility right-of-way, is followed by another 45° horizontal bend to the left, each 45° bend is equal to half of a 90° bend, corresponding to a total of ½ + ½ = 1 full 90° bend; i.e. $n_1 = 1$.

It is considerably more difficult to predict or determine the value of the cumulative unplanned curvature, n_2, since this will obviously vary among installations due to soil conditions, expertise of the crew, … However, the following rule may be used to provide a reasonable estimate for a mini-HDD operation:

$$n_2 \approx L_{bore}\,(\text{ft}) / 500\ \text{ft}. \tag{9}$$

i.e., there may be assumed to be effectively one 90° bend, due to path corrections, for each 500 ft. of path length. This rule is based upon limited experiences, including analyses of sample as-built data provided in mini-HDD equipment user manuals. The above suggested value is consistent with the general magnitude of the corresponding curvature of the actual installed paths. It is noted that this value is not necessarily intended to be a conservative estimate, and that significant variability may be anticipated (see Section 4).

The magnitude of unplanned path curvature provided by Equation 9 is intended to be applicable to a mini-HDD operation, which typically uses steel drill rods of approximately 2-in. diameter. Larger diameter drill rods are stiffer and therefore result in more gradual path deviations and corrections, resulting in a reduced level of path undulations. Thus, when applying the above procedures to a midi-HDD operation, a reduced value of n_2 should be used. In particular, since the rod stiffness is directly proportional to rod diameter, the following general value is implied

$$n_2 \approx [L_{bore}\text{ (ft)} / 500\text{ ft.}] \cdot [2\text{-in} / d\text{ (in)}] \qquad (10)$$

where d refers to the diameter (inches) of the steel drill rod. For example, a 4-in. diameter drill rod would correspond to one 90° bend every 1,000 ft.

The above linear dependence of (unplanned) curvature on rod diameter is consistent with maintaining an equivalent stress level in the steel rod, and corresponds to approximately one-third that typically allowed by bending specifications provided by drill rod manufacturers. Although, in principle, this same rule may be extrapolated to maxi-HDD, using corresponding large diameter drill rods, it is considered excessively conservative for such well-planned, well-controlled installations.

Buoyant weight. In order to apply Equation 7, it is necessary to determine the buoyant weight, w_b , of the portion of the PE pipe submerged in the drilling fluid in route segments L_2, L_3, and L_4, illustrated in Figure 1. ASTM F 1962 provides general formulae for calculating the effective buoyant weight of the pipe under various conditions, including empty, filled with water, and filled with drilling fluid. For the present mini-HDD case of interest, for which the pipe is empty, and, as suggested in ASTM F 1962, the specific gravity of the drilling fluid (mud), γ_b , is conservatively assumed to be equal to 1.5, the buoyant weight may be conveniently determined by

$$w_b\text{ (lbs/ft)} = 0.5 \cdot D^2 - w_a\text{ (lbs/ft)} \qquad (11)$$

where D is the outer diameter (inches) of the pipe. The value of w_a may be obtained from the manufacturer specifications for each pipe of interest (diameter and DR rating).

3.2 Pipe collapse (buckling)

The critical pressure, P_{cr} , as given in Equation 3 may be expressed in terms of an equivalent head (ft) of water, for idealized conditions in which the ovality reduction factor, f_o , and tension reduction factor, f_R , are assumed equal to 1.0. Since the (effective) material stiffness, E, and Poisson's ratio, μ , are dependent upon the load duration, the critical pressure is also dependent upon duration. Table 1 is based upon the corresponding information provided in ASTM F 1962 or related industry information (Petroff, 2006), and is applicable to any diameter HDPE pipe. For MDPE pipe, the tabulated values must be adjusted by a factor of 0.75.

Since the drilling fluid is of significantly greater density than water, the indicated pressure head (ft) values of Table 1 must be reduced by a factor of $1 \div \gamma_b$. The values must be further adjusted (reduced) for possible initial elevated temperature

(ASTM F 1962, Appendix 3) as well as the aforementioned ovality and tensile load considerations.

Table 1
Ideal Critical Pressure (Water Head, ft) for Unconstrained HDPE Pipe (73° F)

Duration	**Pipe Diameter to Thickness Ratio (DR)**						
	7.3	**9**	**11**	**13.5**	**15.5**	**17**	**21**
Short Term	2,316	1,131	579	297	190	141	72
10 hrs	1,236	604	309	158	101	75	39
100 hrs	1,126	550	282	144	92	69	35
1,000 hrs	**1,012**	**494**	**253**	**130**	**79**	**62**	**30**
50 yrs	653	319	163	84	54	40	20

There are two phases to be considered with regard to possible collapse of the pipe. During the installation phase, the 10 hour strength would be appropriate, in combination with anticipated values of f_o and f_R during this period. For the post-installation phase, a 1,000 hour collapse strength is employed as the maximum period during which the drilling fluid applies hydrostatic pressure on the pipe, subsequent to which it is conveniently assumed to thicken and actually provide support against possible earth-imposed loads, or by which time the soil has locally redistributed itself around the pipe to provide some lateral support against buckling. For this post-installation phase, the tension reduction factor, f_R , is equal to 1.0.

Based upon the ovality reduction factor dependency trend provided in ASTM F 1962, and consistent with the present simplified approach, it is reasonable to assume a maximum overall value of f_o of approximately 0.5 to account for ring deformation due to initial ovality plus that induced by installation or post-installation loadings. Such deflections may be induced by bending, aggravated by tension-induced wall bearing pressure, or post-installation soil loads.

The greater collapse strength at 10 hours relative to that at 1,000 hours, would tend to be offset somewhat by the tension reduction factor, f_R , during installation as well as the degradation at possible elevated temperature as the PE pipe sits on the surface prior to installation. However, the latter effects would not be experienced simultaneously, as the pipe is placed at its maximum depth and distance. Therefore, the 1,000 hour (post-installation) collapse characteristics, as adjusted, are conveniently employed to evaluate the vulnerability to collapse. Based upon the above discussion, the following formula results

$$\text{H (ft)} = \text{1,000 hr water head} \bullet f_o \bullet f_R / \gamma_b$$

$$= \text{1,000 hr water head} \bullet (0.5) \bullet (1.0) / (1.5)$$

or,

$$H \text{ (ft)} = 1{,}000 \text{ hr water head} / 3.0 \qquad (12)$$

where H is the allowable head of drilling fluid (i.e., maximum pipe depth).

4. Implementation

4.1 Pull force

Equations 7 – 10 provide a means of predicting the peak pull force, T_D[1], on the pipe during a mini (or midi) - HDD installation. The predicted load should then be compared to the safe pull strength for the pipe, which is provided in Table 2 for HDPE for a variety of pipe sizes. The safe pull strength (lbs) is based upon the safe pull tensile stress (SPS), as given in ASTM F 1962, as applied to the pipe cross-section. The SPS accounts for the load duration, assumed to be 12 hours for the HDD applications, and a significant reduction (less than half) relative to the nominal tensile test strength of HDPE (3,200 lbs/in^2) to limit non-recoverable viscoelastic deformation (Petroff, 2006). For MDPE pipe, the values in Table 2 must be adjusted by a factor of 0.75.

Table 2
Safe Pull Strength (lbs.), HDPE Pipe, 12 hours

Nominal Size	**Pipe Diameter to Thickness Ratio (DR)**						
	7.3	**9**	**11**	**13.5**	**15.5**	**17**	**21**
2-in.	2,400	2,000	1,700	1,400	1,200	1,100	900
3-in.	5,200	4,400	3,700	3,000	2,700	2,500	2,000
4-in.	8,600	7,200	6,000	5,000	4,400	4,000	3,300
6-in.	18,500	15,500	13,000	11,000	9,500	9,000	7,200
8-in.	31,000	26,000	21,500	18,000	16,000	14,500	12,000
12-in.	69,433	58,006	48,538	40,282	35,446	32,515	26,635

ASTM F 1962 requires that the predicted peak tensile load be no greater than the corresponding safe pull strength, without the employment of any explicit safety factor. This is considered reasonable for a typically well-planned, well-controlled maxi-HDD operation since there is a degree of conservatism incorporated into the employed material properties, and in the other parametric values. However, for a typical mini-HDD installation, there may be a wide variability in the as-built route characteristics, such as the degree of actual path curvature and undulations, as discussed in Section 3.1, somewhat less than ideal drilling practices, and other departures from the idealizations and various approximations and assumptions incorporated into the simplifying model described herein. A factor of safety of 2-to-1 is suggested to account for such effects, as well as to indirectly provide additional margin against collapse due to possible degrading effects of tension as discussed in Section 3.2; i.e., it is required that

$$T_D^{\,1} \text{ (Eqs 7 - 10)} \leq \text{ safe pull strength (Table 2) / 2.0} \qquad (13)$$

4.2 Pipe collapse (buckling)

ASTM F 1962 specifies that the effective applied pressure be less than the collapse load as given by Equation 3, but with a safety factor, such as 2-to-1. As applied to Equation 12 for mini-HDD applications, the requirement therefore corresponds to

$$H \text{ (ft)} \leq \text{ 1,000 hr water head (Table 1) / 6.0} \qquad (14)$$

Such additional margin is intended to account for loads or degrading effects not previously directly considered, such as excess ovaling, handling, hydrokinetic pressure, …

This procedure is considered reasonably conservative for mini-HDD installations for drilling under roads or most other obstacles. However, for river or creek crossings, in which the soil loads may be influenced by the overlying water, additional considerations may be warranted, such as discussed in *Polyethylene Pipe for Horizontal Directional Drilling* (PPI, 2000).

5. Comparison to Field Data

The best measure of the validity of the presented simplified model in predicting the pull load on a PE pipe during an HDD operation is a comparison to actual field data. Pull loads for maxi-HDD operations have been reported in the literature (e.g., Francis et al, 2004), but is generally not directly applicable to the present case of interest -- i.e., mini-HDD installations. Furthermore, in many cases the pull force experienced by the pipe is not directly measured by an in-line force gauge, or equivalent, but is determined by means of other information monitored at the drill rig (hydraulic pressure, ...). The latter information reflects not only the resistance or drag experienced by the product pipe, but also the drag forces on the drill string in the ground, as well as the load imposed on the back-reamer. In other cases, the pull force on the pipe itself is deduced from the load at the drill rig based upon a sequence of pull-back operations in an attempt to extract the pull force on the pipe itself (e.g., steel pipe; Puckett, 2003). The ideal field data would be that directly measured by an in-line force gauge at the leading end of PE pipe, as it is installed during a mini (or midi) - HDD operation. Fortunately, such data has been presented in Finnsson (2004) and Knight et al (2002).

Finnsson (2004) provides data obtained during a trial of a commercially available product (TensiTrak™) for monitoring tension at the leading end of the pipe. In particular, a detailed plot of force vs. installed length is provided for a 6-in. DR 11 HDPE pipe installed in a nominally straight, 460 ft. route. The data shows a monotonically increasing tension, reaching a peak load of 3,500 lbs at the completion of the installation. In this case, a midi-HDD drill rig, with 15 ft. long, 3.5-in. diameter drill rods was employed, and Equation 10 (vs. Equation 9) must be used to estimate the unintended path curvature. The following physical properties and characteristics therefore apply:

L_{bore} = 460 ft.

D = 6.625-in.

w_a = 4.7 lbs/ft

n_1 = 0 (no deliberate route bends)

n_2 = $[L_{bore}$ (ft) / 500 ft.] • [2-in / d (in)]
= [460 ft / 500 ft] • [2-in / 3.5-in]
= 0.53 (additional equivalent number of 90° bends)

n = $n_1 + n_2$
= 0.53

w_b = $0.5 \cdot D^2 - w_a$ (lbs/ft)
= $0.5 \cdot (6.625)^2 - 4.7$ (lbs/ft)
= 17.2 lbs/ft

Thus, Equation 6 predicts a peak pull load of

$T_D{}^1$ = $[L_{bore} \cdot w_b \cdot (1/3)] \cdot (1.6)^n$
= $[460 \text{ ft} \cdot 17.2 \text{ lbs/ft} \cdot (1/3)] \cdot (1.6)^{0.53}$
= 3,383 lbs

which is within 3% of the measured load. This degree of accuracy is considered somewhat fortuitous, considering the complicated process and simplified mathematical model that attempts to estimate the associated soil imposed drag loads. Ignoring the term corresponding to the unintentional curvature (i.e., assuming $n_2 = 0$) would result in a predicted tension of less than 2,650 lbs -- underestimating the load by 25%.

Knight et al (2002) describes a series of three installations, using and reusing the same 590 ft. long nominally straight borehole path, pre-reamed as necessary to approximately 50% greater than the outer diameter of the pipe. Two of the installations placed a 6-in. DR 11 MDPE pipe and the third placed an 8-in. DR 17 HDPE pipe, to a depth of 6.5 ft. with entry and exit angles of approximately 11°. The installations were accomplished using 10 ft drill rods, assumed to be of approximately 2-in. diameter, typical for a mini-HDD rig. The recorded peak pull loads were 5,620 lbs., 3,372 lbs., and 5,845 lbs., in the sequence described, and in general were experienced prior to the end of the operations. These loads compare to predicted levels of 5,924, 5924, and 10,580 lbs., based upon the present methodology.

Figure 2 illustrates the results described above, normalized relative to the predicted tension, and demonstrates that the proposed simplified model is able to predict the general magnitude of the experienced peak tensile load during a mini (or midi) - HDD operation, within a factor of 2 or better, based upon the limited sample size. In general, the degree of agreement is excellent, depending upon whether the additional tensile load due to increased curvature from unintended bore path undulations is included in the estimate. In some cases, such considerations result in outstanding agreement, while in other cases the agreement is excellent without considering the additional tension due to this effect. This is not surprising, since the magnitude of such effects will vary widely as a function of the soil conditions, operator skill, and path characteristics. Thus, the simplified model is capable of providing an indication of the general magnitude of the pull force experienced in an actual installation. However, in spite if the demonstrated good-to-excellent agreement in a few sample cases, a wide variability must be anticipated when considering mini-HDD installations in general, due to the complexity of the soil-pipe interaction and associated drag effects, thereby requiring the employment of the design margin of 2-to-1 discussed in Section 4.

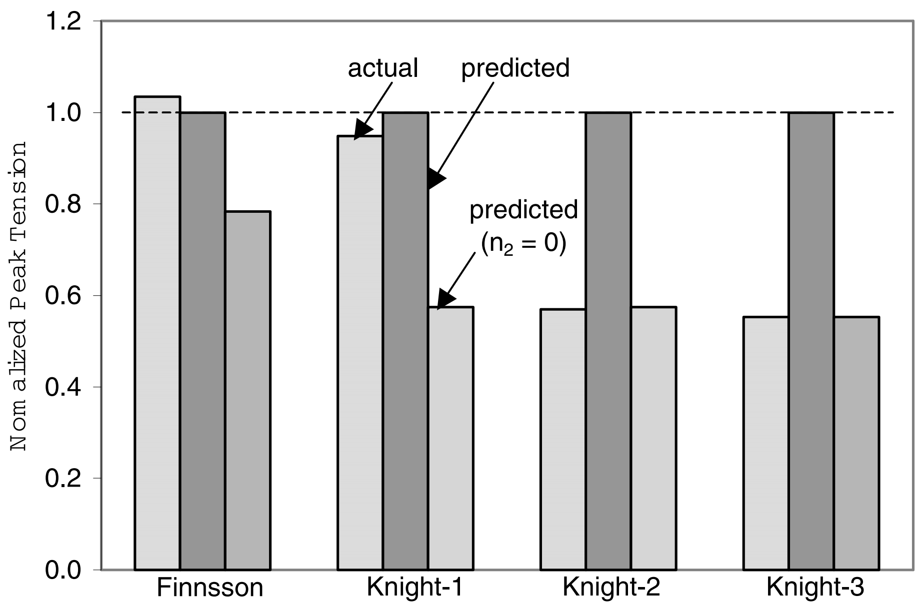

Figure 2. Actual vs. predicted (normalized) tension for mini-HDD installations

When attempting to apply the present simplified method to larger scale (i.e., maxi-HDD) installations, the predicted pull loads tend to be significantly greater than that deduced from the response at the drill rig (Francis et al, 2004). This is not surprising since the present simplified method includes the $(1.6)^n$ term to account for the unplanned path corrections (undulations), which are of considerably lesser magnitude for the well-controlled maxi-HDD operations, as discussed in Section 3.1. However, when the effect of this term is appropriately reduced or deleted, the predictions are in more reasonable agreement with those deduced from the drill rig response. Nonetheless, it is not the intention to apply the present simplified methodology to a large scale, well-engineered maxi-HDD operation. For such applications, the detailed design process, and presumably greater accuracy, associated with more conventional engineering procedures is warranted.

6. Design Example

The appropriate wall thickness of an HDPE pipe, for a given diameter, may be conveniently determined by the application of Equations 13 and 14. As an example, consider the feasibility of installing a 4-in. HDPE pipe of a DR 11 rating, for a relatively long 500 ft mini-HDD route, including one deliberate 90° planar bend, and placed at a relatively large depth of 30 ft.

The following physical properties apply to the HDPE pipe and specified route:

L_{bore} = 500 ft.

D = 4.50-in.

w_a = 2.3 lbs/ft

H = 30 ft

Based upon these values, the following values may be directly calculated:

n_1 = 1.0 (one deliberate 90° bend)

n_2 = L_{bore} (ft) / 500 ft.
= 500 ft / 500 ft
= 1.0 (additional equivalent 90° bend)

$n = n_1 + n_2$
$= 1.0 + 1.0$
$= 2.0$

$w_b = 0.5 \bullet D^2 - w_a$ (lbs/ft)
$= 0.5 \bullet (4.50)^2 - 2.3$ (lbs/ft)
$= 7.8$ lbs/ft

Thus, Equation 7 predicts a peak pull load of

$$T_D^1 = [L_{bore} \bullet w_b \bullet (1/3)] \bullet (1.6)^n$$
$$= [500 \text{ ft} \bullet 7.8 \text{ lbs/ft} \bullet (1/3)] \bullet (1.6)^{2.0}$$
$$= 3{,}300 \text{ lbs}$$

Equation 13 then requires that this predicted installation load, 3,300 lbs, be no greater than half the relevant safe pull strength (nominal 4-in. pipe, DR 11) indicated in Table 2 for HDPE pipe, which corresponds to 6,000 lbs ÷ 2.0, or 3,000 lbs. Although this condition is not strictly met, from a practical perspective, the DR 11 rated pipe is considered sufficient, in recognition of the approximate nature of the calculations and incorporated design margin. Although the example considers a 4-in. pipe, for a given DR value, the predicted pull load, T_D^1 , and the safe pull strength are both proportional to the square of the outer diameter. The conclusions are therefore independent of the pipe diameter. It is noted that the use of the DR 11 pipe in a longer, nominally straight route of almost 700 ft. -- beyond the generally accepted limit (600 ft.) for mini-HDD applications -- would also be predicted to be acceptable, using the present model and criteria.

Regarding the potential vulnerability to collapse, either during or after installation, Equation 14 requires that the peak installation depth, or 30 ft, be no greater than one-sixth the relevant head of water (1,000 hrs, DR 11) indicated in Table 1 for HDPE pipe. This corresponds to a safe depth of 253 ft ÷ 6.0, or 42.1 ft., independent of pipe diameter. Thus, the relatively large 30 ft. proposed installation depth is within the capability of the DR 11 wall thickness.

For DR 11 MDPE pipe, with a reduced tensile capability of 75% that of HDPE, the design placement distances would be less than 400 ft. and 550 ft., respectively, for the case of one 90° planar bend and a nominally straight route. A placement depth of 30 ft would still be considered safe for this type pipe.

This relatively difficult (long, deep) installation(s) demonstrates that a DR 11 HDPE pipe represents a reliable selection for the large majority of mini-HDD applications, and is, in fact, consistent with field experience. Thinner walled pipe (higher DR rating) may be successful in many cases, as may be verified by specific calculations for the route of interest. It is also emphasized that the present methodology for pipe DR selection does not prove that a weaker, thinner-walled pipe would not be successful in practice in individual installations, but -- as in most design procedures -- is intended to serve as a caution that the design may be marginal (non-conservative). Indeed, the relatively long successful installations described in Knight et al (2002), employing DR 17 or MDPE pipe, testify to this possibility.

7. Summary

The proposed simplified methodology attempts to reduce a very complex process, through mathematical simplifications of the theoretical model originally provided in ASTM F 1962 for a maxi-HDD operation, to a readily applied formula (Equation 7) for the purposes of estimating the tensile forces to be experienced installing PE pipe in a mini (or midi) - HDD application. A simplified procedure is also presented to evaluate the potential collapse tendency of the PE pipe during the installation or post-installation phases. The objective is to provide a convenient means of identifying potentially problematic installations and/or to aid in the pipe (DR) selection process. In general, mini-HDD applications do not entail, nor justify, the more extensive planning or analytical investigations characteristic of maxi-HDD projects. The resultant mathematical model reflects the major route parameters (bore length, deliberate bends, ...) and buoyant weight for an empty PE pipe, and also accounts for unintended curvatures (undulations) resulting from path corrections in a typical mini-HDD installation.

A comparison of the predicted pulling tensions to those directly measured, using in-line force gauges, in actual mini- and midi-HDD field installations yields good-to-excellent agreement, providing confidence in the overall procedure for its intended application.

Based upon the application of the methodology to a relatively difficult (long, deep) mini-HDD installation, it is concluded that a DR 11 rated HDPE pipe would be sufficient for most mini-HDD projects. This conclusion is consistent with successful field experiences using this type product pipe.

The specific formulae and methodology are applicable to PE pipe, with its characteristically low stiffness. It is anticipated that future efforts will extend the methodology to pipes with greater stiffness, such as PVC, and possibly steel or iron.

References

ASTM F 1962 (1999). *Standard Guide for Use of Maxi-Horizontal Directional Drilling for Placement of Polyethylene Pipe or Conduit Under Obstacles, Including River Crossings*

Finnsson, S. (2004). *TensiTrak™ – A Tension Load and Drilling Fluid Pressure-Monitoring Device for Horizontal Directional Drilling Installations*, NO-DIG 2004

Francis, M., Kwong, J., Kalani, J., Nakayama, D. (2004) *Comparison of Calculated and Observed Pipeline Pullback Forces During Horizontal Directional Drilling*, NO-DIG 2004

Knight, M.A., Duyvestyn, G.M., and Polak, M.A. (2002) *Horizontal Directional Drilling Installation Loads on Polyethylene Pipes*, NO-DIG 2002

Maurer Technologies, Inc. (1999). *DrillPath*

Petroff, L. (2006). *Designing Polyethylene Water Pipe for Directional Drilling Applications Using ASTM F 1962*, NO-DIG 2006

Plastic Pipe Institute (2000). *Polyethylene Pipe for Horizontal Directional Drilling*

Puckett, J.S. (2003). *Analysis of Theoretical Versus Actual HDD Pulling Loads*, ASCE Pipeline 2003 Conference

Design-Build and Trenchless—a Perfect Solution!

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Abstract

Regardless of the project delivery method (i.e. Design-Bid-Build, Design-Build, etc), trenchless rehabilitation is often overlooked as a solution to rehabilitate underground infrastructure. Pairing trenchless solutions with Design-Build project delivery, however, produces a powerful synergy that is virtually unequalled in construction. Design-Build (D-B) trenchless solutions bring numerous advantages regarding owner and design costs, design and construction schedules, critical path impacts and risk, and project profit that "dig and replace" alternatives do not. Trenchless solutions help allay some of the main owner "trust with verification" concerns while achieving 50-100 year performance. Additionally, trenchless solutions can provide sustainable design benefits that are increasingly important to customers, designers and the environment. The unique characteristics of D-B project delivery fully capitalize on these advantages.

This paper overviews appropriate trenchless technologies and the D-B project delivery. Next, it examines the beneficial relationship between pipeline rehabilitation using trenchless technologies and the D-B project delivery method. It details the criteria for choosing Design-Bid-Build or D-B pipeline rehabilitation from the viewpoints of the owner as well as individual design-build team members. Most importantly, it offers:

- Owners the "right questions to ask"
- Engineers the latest in trenchless applications for design-build
- Design-builders another, high-powered tool for ensuring project costs, schedule success and repeat business

Areas of opportunity and conclusions are presented. The primary position reiterates the beneficial relationship between Design-Build project delivery and appropriate trenchless solutions. The elements are far more successful and powerful when combined, than when trenchless is used in other project delivery methods or when Design-Build utilizes "dig and replace" alternatives.

Discussion

Design-Build (D-B) project delivery system is quickly replacing Design-Bid-Build as the project delivery method of choice, particularly in the water and wastewater markets (DBIA). The majority of these D-B water/waste water projects, however, feature state-of-the-art package treatment facilities while relying upon 19th century "dig and replace" methods of rehabilitating underground water and sanitary and storm sewer lines to renew the supply and distribution system. As a result of these mismatched modern (package systems) and much older ("dig and replace") technologies, the owner and Design-Build team fail to fully exploit the inherent time, cost saving and risk avoidance benefits of D-B project delivery in the Water/Wastewater market. Incorporating trenchless technologies in the D-B water/wastewater project fully leverages the capabilities of both Design-Build and trenchless technologies.

This paper overviews Design-Build-appropriate trenchless technologies and examines the benefits of trenchless rehabilitation of underground infrastructure from three perspectives- the owner, design engineer and constructor. It also outlines the environmental and sustainable design elements that (while often challenging in Design-Build projects) are trenchless advantages. But most importantly, this paper focuses on the synergy that occurs when D-B project delivery is paired with trenchless technologies to rehabilitate existing underground infrastructure, thereby making the project more successful than a D-B project without trenchless…or a trenchless project using Design-Bid-Build project delivery.

Trenchless Solutions

Four families of trenchless technologies including Cured-in-Place Pipe (CIPP), Formed-in-Place Pipe (FIPP), Pipe Upsizing (Pipebursting) and Sliplining are appropriate for Design-Build pipe renewal and replacement applications. Within these trenchless families, the Environmental Protection Agency identifies nearly 30 variations and outlines applications and the state of the technology (EPA). Although trenchless technologies also include horizontal directional drilling and microtunneling, they are primarily used for new pipe installations and, in most cases, provide fewer benefits for the D-B team than the pipe renewal processes.

CIPP is the most widely practiced trenchless technology and is used in rehab projects ranging from 1/2" to 108" in diameter. Although the most common application for CIPP is gravity sewer and storm sewer rehabilitation, reinforced CIPP can be used for pressure water applications up to 80 pounds per square inch. The product can be installed in seamless/jointless lengths up to 3000 feet and millions of feet have been in the ground for over 35 years. CIPP is a fully structural solution with a design life typically exceeding 100 years. The technology consists of a felt tube that hosts a resin system and a curing process (usually steam or hot water).

FIPP material is approved for water applications and perfectly suited for internal pressure applications, such as water supply systems from 2" to 48" in diameter. FIPP typically relies upon the host pipe to carry external loads and features some sort of polyethylene or flexible polyvinyl chloride (PVC) lining technology.

Pipebursting cuts or bursts an existing pipe from the inside and then pulls a welded polyethylene pipe behind the cutter/burster into the enlarged hole. Pipebursting provides a trenchless alternative when increased flow capacity (i.e. a larger diameter pipe) is required. Diameter increases of 0%-25% are commonplace; 25%-50% increases often challenging, and 50%-125% are considered experimental. Water and sewer applications are possible with pipebursting.

Finally, in the sliplining process, a smaller pipe, usually welded high density polyethylene, is pulled or pushed inside a larger pipe. Both water and sewer pipes from 8" to 96" in diameter can be rehabilitated using the sliplining technique.

Owner Benefits

Incorporating trenchless solutions into the Design-Build project provides multiple benefits including:

- lower owner initial and lifecycle costs
- reduced probability of risk-based construction increases caused by weather and environmental issues
- solving owner's "trust with verification" concerns
- long-lived sustainable pipeline solutions
- the opportunity of even more protracted project schedules

Many trenchless solutions provide initial cost reductions of 30-60% and life cycle cost reductions of 30% to 75% when maintenance, 100+ year design life and the potential for replacement are compared to "dig and replace" pipe replacement. For example, an analysis of 2006 CIPP costs for the most typical rehabilitation/replacement project (eight-inch diameter, eight feet deep, 2000-5000 feet long, approximately 350 feet between manholes, service reconnects approximately every 60 feet - typically found in smaller communities with median populations under 25,000) were compared to "dig and replace" with PVC pipe. The analysis assumed the cost of money at 5%, a 100 year CIPP design life and a 100 year PVC design life. Dig and replace construction costs did not account for weather delays, potential remediation resulting from polluted soils or ground water, environmental mitigation measures in environmentally sensitive areas, obtaining permits and rights-of-way, or litigation. Clearly, accounting for these "almost certain" "dig and replace" costs would have considerably improved the CIPP life cycle cost benefit. When Insituform Technologies Inc's actual 2006 bid prices for more than 300 projects were compared to "2006 Means Cost Data" for the same period, CIPP life cycle costs were at least 40% less than "dig and replace" alternatives. (MEANS)

Trenchless solutions also reduce cost and time risks because they are relatively unaffected by the materials they are rehabilitating (e.g. asbestos cement pipe can be relined without removal or remediation) or the polluted soils and ground water they contact via relining. As a result, the need for expensive study, characterization, remediation, project delay and change orders is essentially nil.

Numerous Design-Build Institute of America studies (DBIA) have shown that owners want to ensure they are "getting what they paid for" from D-B projects. In short, owners are willing to trust the D-B team but desire verification. Trenchless technologies provide owners with that assurance. For example, Insituform Technologies, Inc's manufactured products and installation processes are ISO 9000 quality certified. Few, if any "dig and replace" installations provide such assurances of quality. Additionally, the final step in most types of trenchless projects and where direct visual inspection is not possible, is providing a video of the "before and after" pipe rehabilitation. The video of the rehabilitated pipe provides not only a benchmark for an owner's asset and infrastructure management system but also an assurance of proper installation and seamless, jointless flow characteristics. Although trenchless rehabilitation methods, except pipebursting, reduce the inside diameter of the host pipe to some extent, almost without fail, the seamless/jointless, very smooth lining provides improved or equal flow capacity characteristics over the original pipe material.

Finally, trenchless technologies provide a "green design/sustainable design" capability that dig and replace solutions can never provide. Although US Green Building Council's "Leadership in Energy and Environmental Design (LEED)" currently focuses on buildings (as opposed to processes and products) and what occurs in the building envelope (Moore), there are more than a dozen "site development" and "reuse of existing materials" categories in the LEED evaluation system that trenchless squarely addresses. As a result, trenchless technologies provide the ability to improve LEED ratings…at a cost savings. In addition, when evaluated using "Environmental Building News'" (www.buildingGreen.com-EBN) "Five Green Criteria" for buildings and infrastructure materials and systems (i.e. 1. Made from salvaged, recycled or agricultural waste; 2. Conserves natural resources; 3. Avoids toxins or emissions; 4. Saves energy or water and 5. Contributes to a safe, healthy, built environment), trenchless technologies fare well in all applicable categories. This is due to trenchless' low energy and environmental impact during manufacture, transport and installation as well as environmentally superior final performance and processes as compared to "dig and replace" alternatives.

Design-Build Engineering benefits

For the D-B engineering partner, the benefits include reduced design time and project risk, minimal risk to design or inspect, the flexibility of designing when convenient rather than "first…immediately…and under pressure" and a proven, high quality, 22nd century technological end solution.

There are several reasons trenchless solutions are easier to design than "dig and replace" solutions. Most important to the D-B process, a trenchless company is an ideal D-B partner, capable of coaching the entire D-B team in the latest capabilities of a highly specialized industry. The second is that many trenchless companies have been researching, engineering and manufacturing their products to meet tailored design parameters for years. In addition, there are relatively few pieces of design information that trenchless companies require to determine performance and costing parameters. In fact, during the fall of 2006, the author reviewed completed plans and specifications for 52 trenchless projects designed by a range of engineering firms from across the United States. Over 90% of the designs included significant amounts of unnecessary information that unduly added time and cost to the design process. To minimize wasted time, any engineering partner in a D-B project would do well to involve the trenchless company in the collaboration process as soon as possible, and request from the potential trenchless partner design calculations, third-party proof of long term strength and performance testing, ISO 9000 product and installation certifications along with appropriate solutions, and then verify.

For design engineers, there are fewer factors to designing trenchless technologies, and there are multiple design safeguards built into the design and construction process. For example, once the site survey work of establishing project scope, and salient features such as manhole location and invert elevations is complete, much of the trenchless design work is done. Additionally, a review of the "clean and TV" video with the trenchless contractor helps verify the scope and methods of repair. There are no bedding details, tap connections, specification of alternative systems, or involved specifications. In fact, the essential "trenchless" information boils down to: project scope and location, manholes/access points shown (with invert elevations), any required/known point repairs, lengths & diameters and laterals, a source of water and video tapes if available. The conclusion from the author's study cited above was significant. Designing trenchless reduced design effort by more than 40%, when compared to a comparable "dig and replace" design.

Finally, (although the D-B advantage of "fast tracking" construction often places pressure upon design teams to "design ahead of critical path functions) unlike "dig and replace" solutions that are often a very large and long portion of the "critical path," there is no such pressure with trenchless solutions. Trenchless solutions are rarely on the critical path! Design can occur when convenient…rather than to avert a crisis.

Design-Build Constructor benefits

Trenchless benefits can be summarized as avoiding (cost) risk, (schedule) risk, and (performance) risk.

Trenchless processes are almost always significantly cheaper than "dig and replace" alternatives, particularly when above ground construction, underground utilities, great depths, or the potential for remediation are present. Additionally, trenchless companies can lock in prices as quickly as video inspection tapes can be reviewed and evaluated, increasing confidence in a "not to exceed" project cost given at the 30% stage. Further, because trenchless is not dependent upon weather, or the presence of environmental contaminants, project schedule delays, cost increases, and uncertainty due to weather or pollution are virtually non-existent. Simply put, trenchless allows you to "get in, get finished, and get paid".

Trenchless activities are rarely, if ever, on the project critical path and are typically independent of other construction activities. In many cases, the trenchless process is quick enough that bypassing is not required. As there are no "endless" trenches, the impact on other trade operations, job safety and site appearance are all positive.

Finally, performance is ensured by a 50-100 year design life for the range of trenchless technologies, ISO 9000 quality standards for both products and installations, and post video of the completed construction. The seamless, joint less, structurally rehabilitated pipe ensures a satisfied customer and elevates the potential for repeat business.

The Trenchless/D-B Synergies

One may ask if there any downsides to using trenchless in D-B projects, and the answer would be no. There are, however, three key benefits that only trenchless solutions can add to the success of any design-build project.

The most important benefit for the D-B team is that trenchless contractors make great D-B teaming partners, especially when incorporated early in the project planning process. As previously discussed, the trenchless industry is highly specialized and trenchless firms see more underground video in one week than most D-B teams view in a lifetime. The innovations in trenchless products, processes, performance and capabilities have been driven entirely by technology and the advancements in performance and installation have increased exponentially in the last 35 years. For example, as recently as a year ago, the ability to tap water lines from within (rather than digging up the connection) was not possible. Trenchless companies as partners on your D-B team can: verify pipeline condition, advise on solutions, cost and timing, design the solution, manufacture a solution specifically for your application…and install…all without fanfare.

A second benefit for the owner and the D-B team is that trenchless rehabilitation work elements are rarely if ever, on the project's critical path. Trenchless work can usually be accomplished at any time in the project and relatively independent from other work elements. Additionally, trenchless construction is approximately seven times faster than "dig and replace" solutions (without factoring in weather, remediation, change order or discovered" utility delays that accompany "dig and replace"). Of course, the flexibility of work scheduling is a benefit for Design-Bid-Build projects; however, Design-Build's ability to "fast track" construction and expedite project completion makes trenchless solutions a key element to D-B success that far surpasses "dig and replace". As a result, Design-Build is the only project delivery system that can fully capitalize on the schedule, cost and risk avoidance benefits that trenchless solutions provide. Further trenchless solutions are tools that can take Water/Wastewater Design-Build projects to the next level of project cost, time, and performance accomplishments as well as profitability.

The third benefit to the Design Build team is that trenchless solutions rarely require digging or trenching thus avoiding dangerous, obstructive, disruptive, expensive, damaging, environmentally costly, and highly weather-dependent "dig and replace" solutions. Today's trenchless technologies have progressed to allow laterals and service reconnects for water and sewers to be reinstated robotically from the inside of the pipe, virtually eliminating the requirement for digging except to replace sections of fully crushed lines or severe dips. As a result, the work site is safer for workers and potentially the public- thus minimizing the potential for accidents, disruption and litigation.

Conclusion

Design-Build water and wastewater projects that utilize "dig and replace" solutions are comparable to the latest computer using 1980's era computer chips…the project works, but neither cost nor performance is optimized. Design-Build trenchless solutions bring numerous advantages regarding owner and design costs, design and construction schedules, critical path impacts and risk and project profit advantages that "dig and replace" alternatives do not. Trenchless solutions help allay some of the main owner "trust with verification" concerns while achieving 50-100 year performance. Additionally, trenchless solutions can provide sustainable design advantages that are increasingly important to customers, designers and the environment. The pairing of the unique characteristics of D-B project delivery and trenchless solutions fully capitalizes on these advantages.

References

Design-Build Institute of America (DBIA), (2006),
http: //www.dbia.org/fr_industryin.html.

US Environmental Protection Agency (EPA), "Emerging Technologies for Conveyance Systems: New Installations and Rehabilitation Methods, EPA 832-R-06-004, July 2006.

Design-Build Institute of America (DBIA), (2006)
http: //www.dbia.org/pubs/pd_intro.pdf.

Means, R.S., "Means Building Construction Cost Data 2006" 64th edition.

Environmental Building News, http://www.buildingGreen.com/EBN/checklist.cfm, 2006.

Moore, B., (President, National Association of Demolition Contractors) American Institute of Steel Construction, "Leadership in Energy and Environmental Design" February 2005.

HDD in an Urban Environment: the Bellevue Pump Station Force Main Project

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ABSTRACT

Horizontal Directional Drilling (HDD) is a commonly accepted construction method for installation of pipelines throughout the U.S. and abroad. Most HDD installations are performed in rural areas where there are no significant structures under the alignment and where pipe laydown area and construction staging area are plentiful. However, HDD applications in urban settings have also been gaining increased consideration despite its inherent challenges with a built-out environment and sufficient drill side and pipe side construction areas. The Bellevue Pump Station Force Main is one such HDD project that will construct 5,300 feet of 24-inch diameter (ID) sanitary sewer force main just south of downtown Bellevue, Washington. The two main segments of the force main will be installed by HDD using a 32-inch steel casing pipe and 28-inch HDPE carrier pipe. The first HDD segment is a 3800 feet long, 200 feet deep crossing that will be installed under several residences, Bellevue High School, a condominium complex, and a courthouse building with only 650 feet of pipe laydown area. The second HDD segment is a 1000 feet long crossing under State Route 405 with a 70 foot elevation difference that will require a lane closure of a major arterial in Bellevue, WA. Construction is scheduled to begin in May 2007. This paper discusses the pipeline alignment alternatives evaluated, the selection of the final HDD alignment, the final pipeline design for the HDD crossings, permitting and easement issues, and lessons learned during design.

Introduction

The existing Bellevue Pump Station (see Figure 1) is operated by the King County Wastewater Treatment Division and was completed in 1964. Located just south of downtown Bellevue, all flows are currently conveyed through a 20-inch force main and gravity sewer to the Sweyolocken Pump Station, which is located a few miles south. In order to upgrade the existing facility and handle the anticipated future design flows, the Bellevue Pump Station peak capacity will be increased from 11.03 mgd to 13.6 mgd, and a new 24-inch (ID), 5,300 feet long force main (see Figure 1) will be constructed from the pump station to the Eastside Interceptor (ESI), a major regional trunk sewer. This paper focuses on the HDD-related elements of the new force main design.

Figure 1. Aerial View of New Force Main Alignment.

PRE-DESIGN

Force Main Evaluation Criteria

As part of the initial process to evaluate and select the best alternative for the new force main design, ten evaluation criteria were chosen. These criteria were weighted based on the importance of the criteria, and used to determine an overall quantitative score for each of the alternatives evaluated. The ten criteria are as follows (with weighting factor in parentheses): Life Cycle Cost (3), Traffic Impacts (2), Surface Disruptions (2), Easements/Permitting (2), Geotechnical Conditions (3), Impact to Existing Utilities (1), Constructibility (3), Environmental Impacts (2), Operation and Maintenance (2), and Odor / Corrosion Control (2).

Among these ten criteria, four of these criteria (traffic impacts, surface disruptions, easements, utilities) were selected in large part due to the project's location in an urban environment. As will be described later in this paper, HDD construction of the new force main minimized the impacts to these criteria.

Preliminary Geotechnical Investigation

In order to determine feasible construction methods and/or alignments for the new force main, a preliminary geotechnical investigation was conducted in August of 2004 to determine geologic conditions and initial considerations for trenchless technology, excavation and shoring, and dewatering options. Research of existing subsurface explorations, geologic maps, and field reconnaissance determined that about 75 percent of the project area is underlain to a considerable depth by glacially overridden, very dense and hard soils. Relatively shallow deposits of recent fill, alluvium, and peat underlie the remaining 25 percent of the project area. Three geotechnical soil borings were also completed in the project area that suggested that the glacial soils beneath the site consist of glacial till and till-like deposits over a thick sequence of glacial outwash sands.

Preliminary geotechnical investigations indicated that the predominately glacial soils in the area present generally favorable conditions for horizontal directional drilling and/or microtunnel construction due to relatively high shear strengths and low compressibility of the soil. However, these glacial soils can also contain scattered cobbles and boulders and can be highly saturated with high hydrostatic pressures, which can be problematic and needed to be taken into consideration when developing the design and construction approaches.

Alternatives Evaluation

After obtaining the initial geotechnical information, several conceptual level alternative force main alignments were first identified and evaluated in the initial analysis for the best force main alignment from the pump station to the ESI. Numerous construction methods including open-cut installation, Horizontal Directional Drilling (HDD), microtunneling, and conventional tunneling were initially considered for the different alternatives, and conceptual level cost estimates were also generated for these different alignments and construction methods. After this initial evaluation of the conceptual alternatives, two main alternatives (Alternatives 1 and 2) were chosen for additional evaluation (see Figure 2).

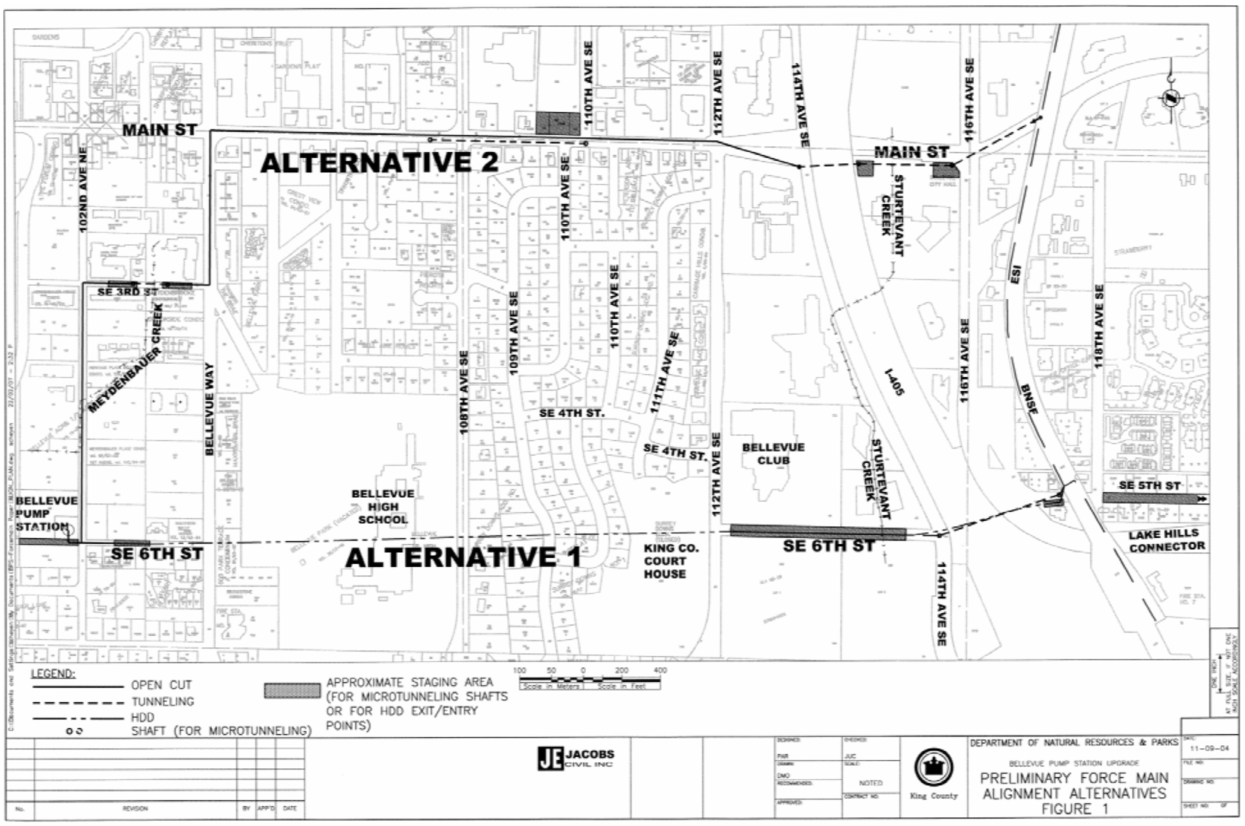

Figure 2. Conceptual Force Main Alternatives 1 & 2.

Alternative 1 is the most direct route and follows the SE 6th Street corridor through several private properties from the pump station to the ESI, utilizing mainly HDD methods for pipeline installation. Alternative 2 is approximately 30% longer, but predominately follows Main Street to connect to the ESI, and utilizes mainly public right-of-way and open-trench installation. Several supplemental analyses (odor control, environmental, permitting, community relations, utility, and hydraulic transient analysis) were also completed in support of the evaluation of these two alternatives. After analyzing these two alternatives with respect to all of the weighted evaluation criteria (see Table 1), Alternative 1 was recommended for further evaluation before a final selection was made for final design.

Table 1. Alternatives Evaluation Matrix.

Evaluation Criteria	**Weighting Factor**	**Alternative 1** (weighted score in parentheses)	**Alternative 2** (weighted score in parentheses)
Life Cycle Cost	3	2 (6)	1 (3)
Traffic Impacts	2	2 (4)	1 (2)
Surface Disruptions	2	2 (4)	1 (2)
Right-of-Way / Permitting	2	1 (2)	2 (4)
Geotechnical Conditions	3	2 (6)	2 (6)
Impact to Existing Utilities	1	3 (3)	2 (2)
Constructibility	3	1 (3)	2 (6)
Environmental Impacts	2	2 (4)	2 (4)
Operations & Maintenance	2	2 (4)	1 (2)
Odor / Corrosion Control	2	2 (4)	1 (2)
Total Score	**NA**	**19 (40)**	**15 (33)**

Single-Reach versus Double-Reach HDD Evaluation

After the recommendation of Alternative 1 alignment from the various force main alternatives, Alternative 1 was further evaluated and revised based on feedback from King County, the City of Bellevue, HDD contractors, and private properties along the alignment. A major issue of evaluation was the installation of the new force main using either a “double-reach” HDD option (two separate and shorter HDD segments, using steel or HDPE pipe for both segments) or a “single-reach” HDD option (one continuous HDD using steel pipe from the pump station to the ESI). The double-reach option consisted of 3800 feet and 1000 feet HDD crossings, with an open-trench segment in between (see Figure 3). The single-reach option (see Figure 4) consisted of a 5300 feet HDD crossing with a potential 2500 feet of pipe laydown area. Although the 3800 feet crossing is shorter in length as compared to the single-reach crossing, this 3800 feet crossing had only 650 feet of pipe laydown area, making the pullback operations more challenging.

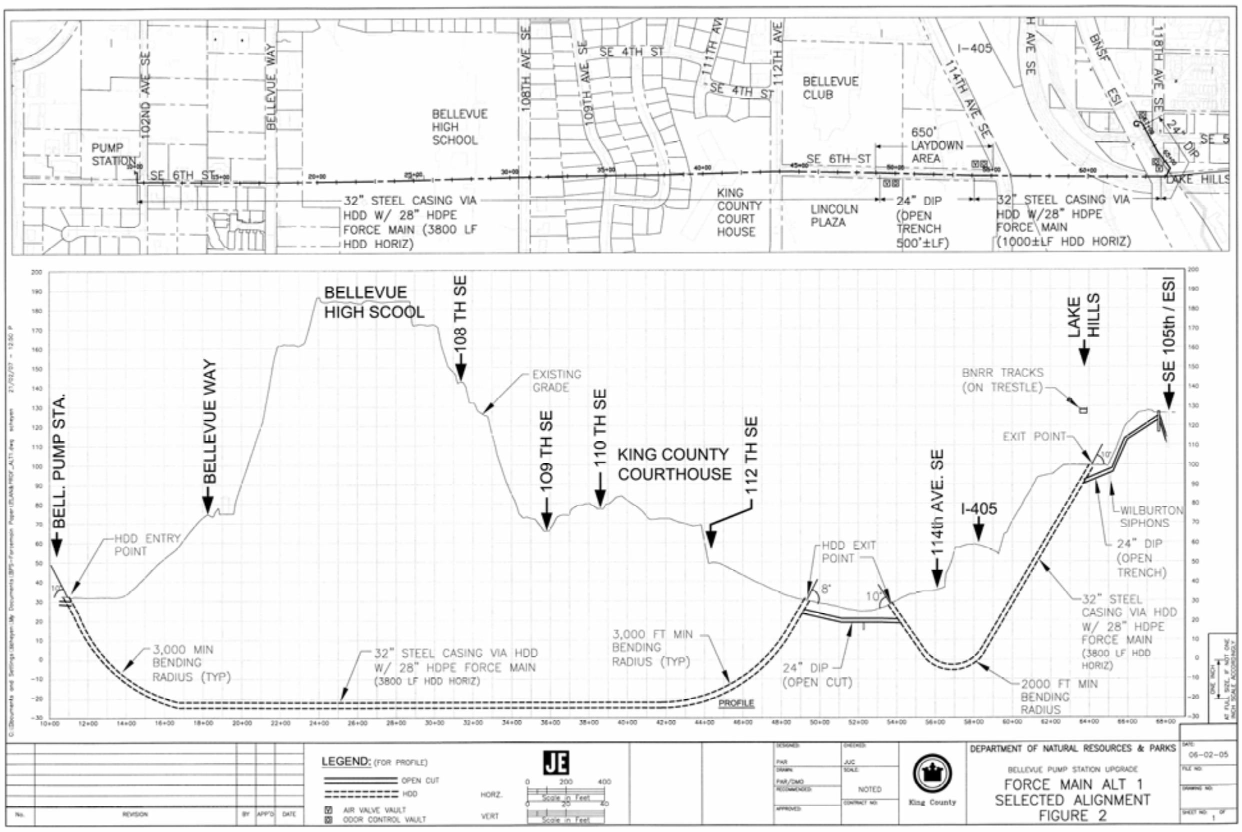

Figure 3. Double Reach HDD Alternative.

In order to evaluate these two HDD options, numerous meetings and site visits were held with King County and HDD contractors to review the options. Detailed technical analyses using HDD design software (DrillPath™) were performed on the two options including a sensitivity analysis of the different design parameters (safety factor, minimum bending radii, wet and dry friction factors, drilling fluid weight, doglegs, etc.). Additional geotechnical borings and analyses (including a preliminary estimate of anticipated boulders along the alignment), pipe materials evaluation, and cost estimates were also completed for these two options. At the conclusion of this evaluation, the single-reach HDD option was initially found to be more technically feasible as compared to the double-reach HDD option with steel or HDPE. This was due to the following:

- Single-reach option seemed to provide a longer laydown area that would require fewer welds (1 or 2 vs. 7 or 8) during the pullback than the "double-reach" option, thus minimizing pullback time and risk of borehole collapse.

- Single-reach option had fewer traffic impacts, surface disruptions, utility relocations, and operations and maintenance requirements since it eliminated construction on SE 6th Street.

- Double-reach option with 3800 feet crossing using HDPE pipe did not meet design criteria for total stress during the pullback.

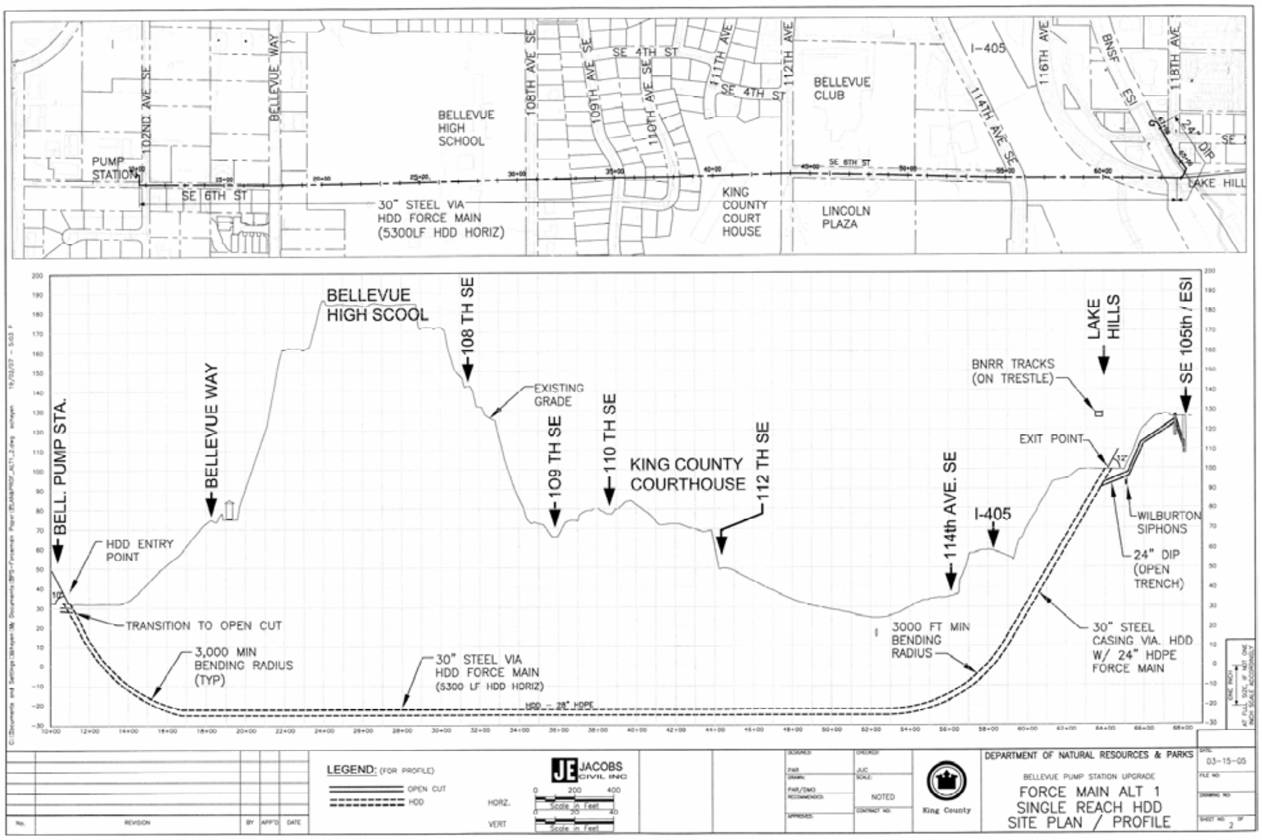

Figure 4. Single Reach HDD Alternative.

Although the initial technical evaluation had recommended the single-reach HDD option, further consideration of costs and research of the single-reach pipe laydown area was needed before a final decision could be made.

Selected Force Main Alternative

Upon further research of the single-reach HDD pipe laydown area, it was discovered that this area had seeping groundwater in some locations, concerns with stability of steep slopes, and wetlands. These features were not clearly evident during the initial evaluation since the area was covered with dense trees and minimal field reconnaissance had been performed. In addition, this single-reach option had a significantly higher estimated construction cost (over $1 million) than the double-reach option, and would require a more extensive permitting process due to the presence of the wetlands.

Therefore, the single-reach option was dropped from further consideration, and the double-reach HDD (with steel casing pipe for the 3800 feet crossing) was selected for final design. Although utilizing steel pipe for the 3800 feet crossing will necessitate numerous welds during the HDD pullback, preliminary soil borings have indicated favorable conditions for hole stability and the strength of the steel pipe will allow contractor flexibility in "un-sticking" the pipe if necessary. Thus, at the end of the pre-design phase, the selected Alternative 1 for the new force main consisted of the following two HDD segments:

- A 3800 feet HDD crossing (30-inch steel casing with 24-inch HDPE carrier pipe) from the pump station, under numerous structures and residences, to SE 6th Street.
- A 1000 feet HDD crossing (36-inch HDPE casing pipe with 24-inch HDPE carrier pipe) under State Route 405 to Lake Hills Connector.

FINAL DESIGN

Final HDD Pipeline Design

During the final design phase, several revisions were made to the final HDD pipe crossings. First, for the 1000 feet HDD crossing, the casing pipe material was changed from HDPE to steel due to Washington State DOT (WSDOT) permit requirements for installing the force main under Interstate 405. This change was requested by WSDOT due to a future highway improvement project that would involve numerous piles being driven in the vicinity of the force main. Both WSDOT and King County agreed that a steel casing pipe (rather than an HDPE casing pipe) would provide better protection to the HDPE carrier pipe in the event that the future pile driving operations inadvertently strike the force main.

Second, final design calculations were completed to compare the ability of the HDPE carrier pipe to withstand different load combinations under various scenarios and to select the optimum combination to install the HDPE pipe and provide a force main that will last through the design life (Year 2050). As a result of this process, the HDPE carrier pipe was changed from 24-inch DR 17 HDPE to 28-inch DR 11 HDPE for the 3800 feet crossing and 28-inch DR 9 for the 1000 feet crossing. Due to the increased diameter of the HDPE pipes, the steel casing pipe was also increased from 30-inch diameter to 32-inch diameter for both HDD crossings.

HDPE Pipe Design Process

During the pre-design phase of the project, the 24-inch HDPE pipe was planned to be SDR 17 and the annular space between the steel casing and the HDPE pipe was to be grouted. The grout would provide a stronger pipe system that could support the anticipated loads. The diameter of the HDPE pipe was determined based on design flow criteria, which required that the internal diameter be an average of approximately 22-inches.

At the 30% design stage, an independent value engineering (VE) team submitted a recommendation to the project team to eliminate the annular space grout to reduce the cost and risk of construction. The VE team believed that grouting a 3,800 feet long alignment and confirming that grout completely fills the annular space could be difficult. In addition, if the grout begins to set before completing the installation, there would be limited options to remedy the situation and to provide the soil support that the force main design had taken into account. In addition, a pressure surge (transient) analysis completed after the 30% submittal indicated that the maximum pressure in the force main was predicted to reach levels beyond the internal surge capacity of the DR 17 HDPE pipe if it was grouted. Since the HDPE pipe would have been grouted inside of the steel casing, the analysis had modeled the HDPE pipe as a stiffer pipe material that in turn generated higher wave speeds and higher pressure surges. If the grout were eliminated, the HDPE pipe would have lower wave speeds and lower pressure surges. Therefore, the annular space grouting was eliminated from the pipe system design.

Since the grouting was eliminated, the HDPE pipe design could not account for the soil support that would have been provided by the grout. This condition would be mitigated if the steel casing pipe does not corrode for the duration of the design life of the project (2050). If the steel casing pipe remains intact through the design year 2050, then the HDPE pipe would only need to support the external hydrostatic loads from the groundwater (assuming pinholes in the casing pipe), and the steel casing pipe would continue to support the earth loads above the casing. However, if the steel casing does not remain intact, then the HDPE pipe would need to support the external groundwater as well as the earth loads above the deteriorated steel casing pipe.

Therefore, a corrosion analysis was completed for the 30-inch steel casing pipe using information collected from the soil borings performed earlier. This analysis indicated that certain portions of the steel casing pipe would not last up to the design Year 2050, and could possibly only last approximately 17 years for worst case conditions. Options considered in protecting the steel casing pipe included a galvanic (sacrificial) anode system, an impressed current cathodic protection system, galvanizing the casing pipe, and coating the casing pipe with fusion-bonded epoxy. However, none of these options were recommended due to prohibitive costs or uncertainties in the effectiveness of these options in an HDD application.

Finally, it was decided to design the HDPE carrier pipe to handle the permanent external loads (earth and groundwater) and internal loads (operating and surge pressure) with the assumption that the steel casing pipe would corrode away under worst case conditions. This way, the force main would continue to operate regardless of what happened to the steel casing pipe. The final analysis and calculations determined that a 28” DR 11 HDPE pipe for the 3800 feet crossing and a 28” DR 9 HDPE pipe for the 1000 feet crossing provided the most cost-effective and technically sound solution in meeting the project’s design criteria.

ADDITIONAL CONSIDERATIONS

Final Geotechnical Investigation and Recommendations

Five soil borings were performed during the pre-design phase. 12 additional borings along the selected alignment and profile were performed during the final design phase. These soil borings confirmed that the majority of the force main alignment will encounter hard glacial soils, with some areas of fill and peat/alluvial deposits, along the 5300 feet alignment. In addition, the geotechnical engineer recommended the use of steel conductor sleeves and annular space grouting at the HDD entry/exit points as potential methods in minimizing borehole instability, hydraulic fracture, and ground settlements. This was critical since the entry/exit points were located on urban streets with many nearby properties and utilities, poor soils, and/or heavy traffic volumes.

Construction Staging Area / Street Use Permit Requirements

Since the project is located in an urban area, gaining sufficient construction staging area for the HDD drill side and pipe side operations was extremely challenging. The construction staging area required for the 3800 feet HDD crossing will be on SE 6th Street at both ends of the crossing (see Figure 2). At the anticipated drill side of the crossing near the pump station, SE 6th Street is 24 feet wide and will be closed for a length of 220 feet. At the pipe side of the crossing near the Bellevue Club, SE 6th Street is 36 feet wide and will be closed for a length of 650 feet. Since the total length of the HDD crossing is 3800 feet, there will be at least six welds required during the pullback, but there could be more depending on the location of where the weld is made and the "breakover" distance of the steel pipe as it enters the borehole. Another challenge in this pipe side staging area will be the staging of the welded steel pipe segments as it is welded and prepared for the final pullback. Nonetheless, even with the numerous welds required during the pullback and the limited staging area, the decision was made to select this 3800 feet crossing option due to the presence of favorable soil conditions and the use of steel pipe that could be "unstuck" if problems arose during the pullback.

The construction staging area at one end of the 1000 feet HDD crossing is the same area on SE 6th Street used for the pipeside operations of the 3800 feet crossing. The other end of the 1000 feet crossing will require a staging area along a major arterial in Bellevue (Lake Hills Connector) that will close one of the two heavily traveled lanes. It is anticipated that the HDD contractor will use this area along Lake Hills Connector to prepare and weld the steel casing pipe for pullback since there is enough length to weld the entire 1000 feet crossing prior to pullback.

Early communications with the City of Bellevue was critical in determining what street or lane closures would be allowed by the City at the different entry/exit points and staging areas. Challenges in providing these staging areas in an urban environment included: maintaining or providing alternate access for fire protection services; providing alternate pedestrian routes through or near the construction zones; and maintaining or providing alternate access for businesses along these staging areas. Through a mutually cooperative process, a City of Bellevue Street Use Permit and a Fire Protection Permit was granted by the City for use of these staging areas. However, the Street Use Permit did contain specific durations for how long the streets or lanes could be closed and also contained a liquidated damages clause in the event the contractor does not meet these requirements.

Easements / Agreements

The new force main alignment is located under numerous large buildings, residences, a 10-lane highway, and a BNSF railroad bridge. This is largely because the SE 6th Street right-of-way is not continuous within the force main alignment. In all, 10 subsurface easements and 3 permits were acquired for these properties. Three temporary construction agreements and 5 rights-of-entry were also required for properties requiring settlement and vibration monitoring, additional construction staging area, or alternate access where existing driveways will be temporarily closed during construction. Despite the challenges in obtaining these numerous easements and agreements, all of the documents were successfully completed and condemnation was not required for any of the properties. A key factor in obtaining these documents was in the finalization of the force main alignment at the 30% design stage to maximize the amount of time needed for appraising the properties, communicating and negotiating with the owners, and executing these documents.

Noise Control

In general, noise generated from construction of the force main must comply with the Bellevue Municipal Code 9.18 (Noise Control). Although sounds created by construction are exempt from the provisions of Code 9.18 between the hours of 7:00 a.m. and 6:00 p.m. on weekdays, and 9:00 a.m. and 6:00 p.m. on Saturdays, King County decided to implement stricter noise control standards since construction will be in close proximity to residents and businesses. In order to estimate what noise levels could be anticipated from the HDD construction, an acoustical consultant was tasked to model the anticipated noise levels and to provide mitigation measures that would meet the City and County noise control standards. Based on the results of this analysis, a noise wall was designed at the construction staging area near the pump station since this area was closest to sensitive residents. Other noise control features were also stipulated in the contract documents as the responsibility of the contractor including a noise control and monitoring plan (most likely including fixed noise barrier walls, portable noise barriers, equipment mufflers, noise monitoring, specifications for maximum permissible noise levels from construction equipment, etc.).

Community Relations

Since the project was located in an urban area with numerous residents and businesses along the alignment, community relations were a key element of the project. As such, the goal of community relations was to support successful implementation of the Bellevue Pump Station project by identifying and addressing community concerns and issues. To help meet this goal, a Community Relations Plan was prepared that identified community relations objectives for each project phase (pre-design, design, and construction), target audiences for the project, potential concerns and messages, and summarized public involvement activities. In summary, these activities included setting up a project web site and construction hotline, preparing and distributing public information fliers during geotechnical borings, holding small group meetings with affected local businesses and property owners, and holding public meetings at key milestones during the project.

Risk Management

Managing the risk elements proved to be extremely challenging during the project. Since the new force main needed to be designed in an urban environment, the potential risks were also higher (e.g., settlement under structures, frac-out on streets or I-405, minimal staging areas, liquidated damages for delays to opening City streets, delays due to numerous required easements and permits, several residents and businesses in close proximity to construction areas, etc.). In order to mitigate these risks, a risk analysis process was implemented. This process involved conducting a Pre-Design Risk Workshop to identify risks that may impact the project and documenting these risks on a risk register. Risk probabilities and impacts were assigned to the risks, and the risks were prioritized by risk ranking (high, medium, and low) in order to identify and focus on the key risks to be addressed and monitored at major milestones of the final design phase. As one element of this process, a modified lump sum contracting method was selected to reduce the risk to the contractor by allowing payment at key milestones during the HDD process (mobilization and site preparation, pilot hole, reaming, pipe installation, and demobilization) as compared to the more traditional "no hole, no pay" type HDD contract. However, even with this contract structure that proved to be successful on a previous County HDD project, the risks associated with the Bellevue Pump Station Force Main project contributed to a substantially higher bid price than the engineer's estimate.

CONCLUSION

Designing an HDD project in an urban environment presented numerous challenges in completing both the technical elements and the associated non-technical elements. Despite these challenges, a design was successfully completed through the efforts and collaboration of numerous project stakeholders. Throughout the design process, many strategies for a successful design were implemented as well as many lessons were learned, and are presented as follows:

- Involve major project stakeholders early in the pre-design phase to identify key issues and any "fatal flaws".
- Obtain HDD contractor feedback early when establishing pipeline alignment, pipeline material, and staging area requirements.
- Soil boring information is critical in finalizing a proposed horizontal and vertical alignment. Obtain rights-of-entry and/or permits as soon as possible once a preliminary alignment is selected.
- Research requirements of potential construction staging areas when determining HDD crossings. Although the crossing may be feasible from a pipe stress analysis, there may be fatal flaws in the potential pipe side or drill side staging areas.
- Determine as early as possible if annular space between carrier pipe and casing pipe will be grouted. This may have impacts on the surge pressures and required wall thickness design of the carrier pipe.
- Communicate early and often (at key milestones) with potentially affected residents and businesses to develop trust between both parties. This may provide benefits during the easement negotiations process and when obtaining permits requiring a public hearing process.
- Consider incorporating risk-sharing elements into the project contract structure (e.g., sharing of liability for damage to existing facilities, daily rate contract with incentives for early completion, cost-plus bid item for inadvertent returns, etc.) for complex and higher-risk projects. If risks are shared between the owner and contractor and the risk is placed on the party that can best handle the risk, this will encourage more bidder participation and lower bid prices.

- Confirm crew sizes and equipment from HDD contractors' feedback when preparing cost estimates during design. Although contractors may be hesitant or reluctant to divulge too much information regarding their means and methods, this type of information from contractors can play a key role in the accuracy of the cost estimates and the owner's budgeting process.

Project Status

Final design was completed in November of 2006, and the bid documents were advertised on December 5, 2006. Only one bid was received and opened on January 18, 2007. After a thorough bid evaluation process, the contract was awarded to Michels Directional Crossings. Construction is scheduled to begin in April 2007.

References

Flow Science. (2006). *Bellevue Pump Station 90 Percent Pressure Surge Analysis*, Pasadena, California.

Jacobs Engineering Group. (2005). *Bellevue Pump Station Upgrade Pre-Design Report*, Seattle, Washington.

Norton Corrosion Limited. (2006). *Soil Samples Corrosivity Testing – Bellevue Pump Station Force Main*, Woodinville, Washington.

Olympic Associates Company. (2006). *Bellevue Pump Station Force Main Peer and Constructibility Review Report*, Seattle, Washington.

Shannon & Wilson. (2005). *Bellevue Pump Station Upgrade Project Geotechnical Data Report – Force Main*, Seattle, Washington.

Planning Methodology for Small Diameter Pipelines in an Urban Environment

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Abstract
Buried municipal infrastructure is rapidly expanding due to increased urbanization. Subsequently, engineers and contractors are faced with having to designate the most effective method of installing new buried infrastructure while being mindful of cost considerations. In order to determine the most feasible underground construction method, various risks and cost factors must be assessed and analyzed. A comprehensive survey was conducted at Arizona State University to solicit input from twenty-eight contractors in various geographical regions throughout the United States and Canada. The intent was to determine specific risk factors and their inherent impacts, as well as to quantify cost comparison between Horizontal Directional Drilling (HDD) and traditional open cut construction methods. This paper identifies and describes risks and cost factors that were subsequently used to create a planning tool for small diameter pipelines in an urban environment.

INTRODUCTION
As the density of urban and suburban neighborhoods increase, so does the congestion of underground infrastructure making open cut construction methods more difficult and dangerous. In recent years, alternative construction methods have become more important for municipalities to consider. The goal of municipalities is to look for more feasible methods to reduce costs and improve safety. Horizontal Directional Drilling (HDD) is an applicable and alternative method that can be used in urban environments for pipe diameters less than 24 in. (600 mm). HDD is desirable because of its ability to maneuver around existing utilities and subsurface objects. With more utilities being installed in developed areas, underground infrastructure has further increased costs and risks of digging up roadways, going under major pipe lines, water channels and underground utility corridors. The main influential factor for owners and municipalities is the cost of a project; however, eco-social effects, construction consideration factors and risk factors are becoming increasingly more important for selecting the most viable construction method.

As with many construction projects, the difficulty lies with the economical reasoning for project funding. Municipalities invest considerable amounts of funding for sewer and water infrastructure as their communities mature. The most common method used for underground utility construction is traditional open cut construction due to the simplistic approach of excavating soil. Generally, when open cut is not acceptable or desirable, HDD practices can be used. In situations with high investments in surface infrastructure, congested existing utilities, and where social costs such as commuter traffic and businesses are affected; HDD is a more desirable choice (Lueke and Ariaratnam 2005). Low environmental and social impacts, safety issues and soil conditions also make HDD a desirable method for underground construction. Further information and specific details on the HDD process can be found in Knight et al. (2001) and Kramer (1992). Through this research, it is expected that industry personnel a better method of choosing between HDD and open cut can be established, and those who are new to the industry can develop a better understanding of the many benefits HDD offers.

RESEARCH OBJECTIVES AND METHODOLOGY

The main objectives of this research were to identify and gather cost differences between HDD and open cut construction from industry specialists and identify factors that help indicate the most desirable construction method for a project. Our main focus was to investigate the installation of pipes and conduits ranging in size from 2 in. to 24 in. (50 mm - 600 mm) diameters. We distributed a survey to 28 contractors with experience in HDD and open cut construction. The seven-sectioned survey questionnaires were distributed across the United States and Canada to obtain diverse types of situations and conditions. Approximately 61% of the respondents were located in the United States and the remaining 39% were located in Canada. In the first two sections, questions relating to the contractors' business activities and project information, such as typical project sizes and project materials, were gathered. Section three of the survey asked contractors to provide the most common types of contracts and rank factors that significantly contribute in determining a bidding estimate and estimating a contingency, on a scale from one to ten. Section four of the survey addresses a comparison for cost factors, through percentages, found in underground utility projects that are able to be constructed using HDD or open cut. In section five of the survey, a direct comparison of eco-social factors for HDD and open cut was conducted. Section six pertains to consideration factors affecting the type of construction selection for a project that could be completed with either method. The final section of the survey was used to identify the five most important factors when considering HDD or open cut on a project. Respondents were asked to give their opinion on cost differences and myths that would prevent HDD from being used. Also included in the final section of the survey was a question regarding the knowledge level of owners and engineers in the underground construction industry.

DATA ANALYSIS

Profile of Contractors

Feedback from 28 contractors in the United States and Canada was analyzed. From the companies surveyed, approximately 71% performed both HDD and open cut construction. The remaining 29% performed only horizontal directional drilling.

Different types of construction that the contractors performed are displayed in Table 1. The most common type of construction is Water/Sewer/Storm at 89%. From these respondents, approximately 4% performed only Water/Sewer/Storm while the rest performed other types of services as well. The second most common type of construction is underground utility installation at 86% of the respondents.

Table 1. Percent of respondents for areas of specialized work

Types of Specialized Work	Percent of Respondents (%)
Water / Sewer / Storm	89
Underground installation (phone, fiber optics, electrical, etc.)	86
Pipeline (oil, natural gas)	61
Environmental Remediation	36
Horizontal Sampling	21

A range of annual revenues for contractors who participated in the survey have been categorized and are shown in Figure 1. Approximately 21% of the companies surveyed had annual revenues less than \$2 Million (M). These companies can be categorized as smaller HDD subcontractors with approximately 2 to 13 permanent employees. Twenty-nine percent of the respondents identified themselves with revenues between \$2 and \$4 M and these companies generally have 7 to 27 full time employees. Seven percent of the contractors surveyed had revenues ranging from \$4 to \$8 M and had 15 to 27 permanent workers. Companies between \$4 and \$8 M do not have a significant difference in the number of employees compared to companies ranging in size of \$2 to \$4 M. A large number of employees are evident with companies ranging at \$8 to \$14 M with approximately 100 to 115 workers; this group of companies comprised 7% of the respondents. The largest category surveyed was those with annual revenues over \$14 M, and the permanent number of employees was drastically larger, ranging from approximately 45 to 1,300. This large portion of companies ranging from \$14 M plus was not anticipated to be the majority (36%) of the respondents. However, a survey conducted by Allouche et al. (2000) found that 23% of the contractors surveyed had annual revenues greater than \$14 M. These large companies can be categorized as national or international dedicated directional drilling contractors or underground construction divisions of more diverse construction companies.

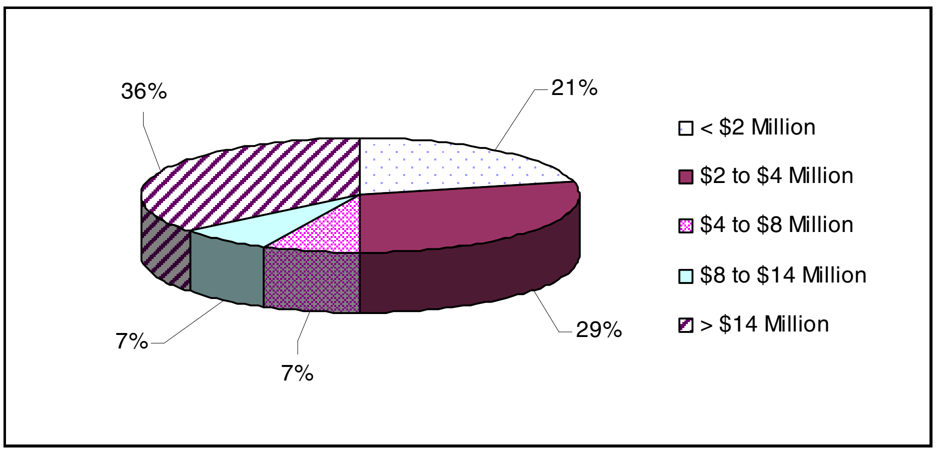

Figure 1. Percent of respondents in each range of company's annual revenues (in millions of U.S dollars)

Section two of the survey, investigates the breakdown of work obtained through competitive bidding by percentage. The results of the survey indicated that approximately 39% had between 75-100% of their annual work acquired from competitive bidding. Table 2 indicates the rest of the results regarding work obtained from competitive bidding.

Table 2. Percent breakdown of annual work obtained by contractors through competitive bidding

Percent of Work Obtained Through Competitive Bidding (%)	Number of Contractors	Percent Obtained (%)
< 25 %	2	7
25 – 49 %	7	25
50 – 74 %	8	29
75 – 100 %	11	39
Total	28	100

A breakdown of the price range for projects by percentage was conducted to see what kind of project sizes were being constructed in 2005-06. The most common price range for underground construction projects of the contractors surveyed was from \$180,000 to \$350,000, making up 24% of the respondents. Overall the percentage breakdown for the prices of projects is found to be fairly consistent throughout the ranges as seen in Table 3.

Table 3. Percentage of respondents within each average project cost range

Project Dollar Value	Percentage of respondents (%)
< \$7,000	1
\$7,000 - \$18,000	16
\$18,001 - \$35,000	8
\$35,001 - \$70,000	12
\$70,001 - \$80,000	16
\$180,001 - \$350,000	24
> \$350,001	14
Total	100

Contingency Plans

The final question within section three, asked the contractors to rank on a scale from 1 to 10, factors they consider to be the most important when estimating an underground utility project (1 being least important and 10 being most important). These results are displayed in Table 4, in descending order of priority. The contractors' previous experience was by far the most important factor when estimating a contingency plan. As with any underground construction, previous experience is always a critical aspect to the technical and financial success of a project (Allouche et al., 2000). As contractors become experienced with their drill rig or backhoe capabilities under various environments; productivity will increase and contingency needs will be reduced. Availability of soil data was another important contingency factor identified by the surveyed contractors. With more information on soil conditions, a reduction in the overall bid prices will occur. Other factors that were received from a few survey respondents but not included in the results were: seasons; design restrictions; drill setup; pipe layout locations and prevailing prices for the market. These additional factors would give more specific information regarding a contingency plan if they were included in the survey; however, most respondents did not identify them.

Table 4. Relative importance of factors in estimating a contingency plan

Factors for a Contingency Plan	Score
Previous Experience	8.3
Availability of soil data	7.9
Type of Contract	6.8
Location (Urban/ Rural)	6.8
Project Size	6.5
Owner/Client	6.5
Proximity to home base	5.6

Price of a Project

In order to determine the cost for a typical project, six factors were examined in order to identify the most important part of the overall price. Table 5 presents survey results of factors that affect the bid price of a project. The type of construction method was found to be the most important factor in determining the bid price for a project. Contractor's previous experience and quality of the cost estimate was a close second. The number of competitors did not seem to have a large affect on the overall price of the project, which implies that contractors must be giving competitive bids to the owners. Some other factors received from survey respondents include: seasons; proximity to office; and the engineering firm. Once again, only a few respondents mentioned these additional factors. As a result, we assumed they were not as important as the ones listed in the survey.

Table 5. Relative importance of factors in estimating a bid price

Factors Effecting Bid Price	Score
Construction Method	7.8
Experience	7.7
Quality of your cost estimate	7.7
Current work load	7.5
Owner/Client	6.5
Number of competitors	5.8

Section four of the survey was created to determine typical costs for an underground construction project. This was by far the most difficult part of the study. Some contractors claimed that it was not possible to compare costs of HDD with open cut, while others seemed to respond with encouraging feedback when comparing with percentages. The cost comparison results between HDD and open cut are displayed in Figure 2. This figure shows various cost factors that contractors considered a savings for HDD, designated by a positive percentage. A negative percentage indicates a savings realized by open cut. Two-thirds of the factors observed in the survey were identified to be a savings for HDD. From these results, we were surprised to see that equipment operation costs for HDD were not considered to be a savings factor. The factor with the most notable controversy between respondents was equipment operation cost. Some of the respondents claimed that only the initial upfront cost of a drill rig is the biggest expense. Another reason for HDD's higher equipment cost is the maintenance after drilling through difficult conditions. Duration on an HDD project was found to have a time savings of 16% over open cut. Time savings can be a significant factor on a project if ground water or saturated soils are evident. Additional profit on an HDD project was found to be approximately 12%. Surface restoration costs are the most significant cost savings that HDD offers. The savings in restoration for paving alone was near 70%. Job site management and the operational cost of labor have a slight savings for HDD; however, both factors are less than 10% savings.

Three of the cost factors considered indicated open cut to be have a slight savings over HDD. The averages of the respondents indicate that contractors spend approximately 9% more on equipment operational costs for HDD than open cut. Material costs for HDD were also found to be nearly 3% higher. The engineering service costs on a project are expected to be similar for HDD and open cut.

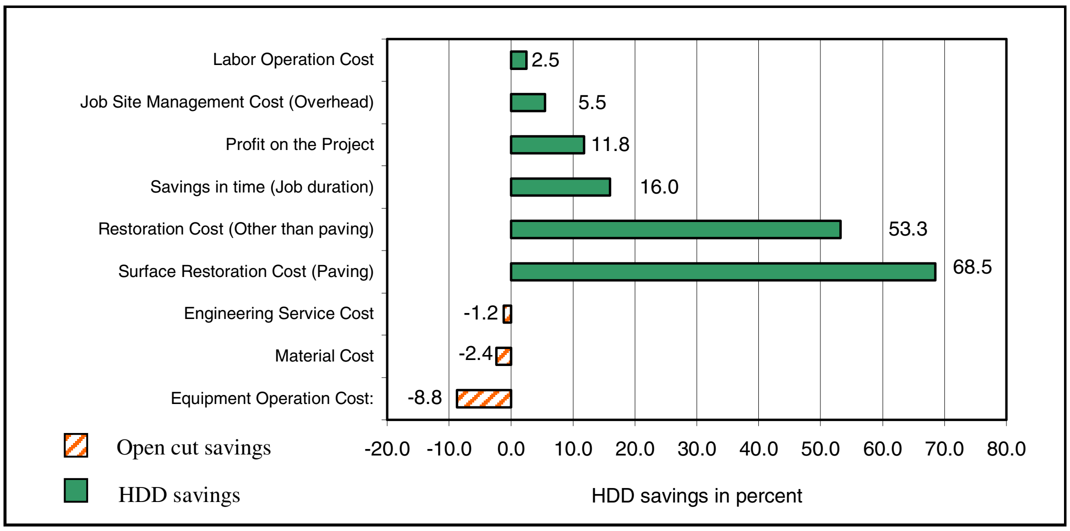

Figure 2. Percent comparison of cost factors between HDD versus traditional open cut (negative percent indicates savings realized by open cut)

Eco-Social Impacts

Eco-social factors were examined in section five of the survey by using a scale ranging from 1 to 10. Results indicate that HDD significantly reduces a project's environmental impacts (Table 6). Dust pollution on an open cut project was found to be almost three times the impact than on an HDD project. There are numerous advantages when HDD is used relating to decreased traffic disruption, less effect on business sales, and the decreased impact on the ecological system. Also, HDD scored higher for removal of waste materials. This is due in large part to the difficulty of finding a designated area to discharge the drilling fluid. On open cut projects, if soils are removed from the site, disposal of waste materials is usually non-problematic if the soil is not contaminated.

Table 6. Relative importance of eco-social impacts to construction methods

Eco-Social Impacts	Score		
	HDD	Open Cut	Difference
Dust Pollution	2.4	7.2	-4.8
Travel effect on general public	3.4	7.7	-4.3
Effect on the ecological system	3.2	6.8	-3.6
Vibration	3.2	6.3	-3.2
Effect on business sales	4.3	6.4	-2.1
Noise Pollution	4.9	6.6	-1.7
Operational costs by contractor	5.4	6.4	-1
Maintenance costs by contractor	6.9	6.1	0.8
Disposal of waste material	6.6	4.6	2

Consideration Factors

Similar to the eco-social section of the survey, consideration factors were compared directly between HDD and open cut on a scale of 1 to 10. Results from the comparison are shown in Table 7, which indicates the importance of having a detailed understanding of the soil conditions. Having the proper information and quantity of existing utilities in the construction area is as critical in open cut as it is in HDD. As would be expected, the ground water table and the weather conditions are much more critical on an open cut project. The impact of surface obstructions for open cut is much more important. However buried obstructions such as timber and concrete have an importance for HDD. Safety issues for both types of construction were surprisingly similar, even though HDD was expected to have a lower rating due to a significant reduction of open trenching and a reduction of fatalities.

Table 7. Relative importance of project consideration factors to method

Project Consideration Factors	Score		
	HDD	Open Cut	Difference
Surface Obstructions	5.1	8.6	-3.5
Ground Water Table	5.2	8.5	-3.3
Weather Conditions (Rain/Snow/Heat)	4.8	7.3	-2.5
Traffic Restrictions	5.7	7.8	-2.1
Safety Issues	7.4	8.0	-0.6
Density of Existing Utilities	8.5	8.3	0.2
Availability of Existing Utilities Info.	8.7	8.0	0.7
Buried Obstructions (i.e. timber, concrete, etc)	7.6	6.5	1.1
Soil Condition / Properties	8.5	6.9	1.5

Potential Risk Factors

Table 8 shows the top five most significant factors to consider prior to construction of a HDD project. All responses were categorized into 13 different sub-headings and each heading was then accumulated. Approximately 68% of the respondents indicated soil conditions were the most important factor to consider before HDD is selected as a viable method. This result was not surprising because the most challenging situations that contractors encounter during HDD are gravels and cobbles (Ariaratnam et al., 2004). The second most important factor was site access at 46%. Having adequate site access can help reduce wasted time spent on material handling and equipment maintenance and mobilization. Traffic, which can also be classified as a social factor, was another important consideration on a project.

Table 8. Contractors' Top Five Factors for HDD

	HDD Factors	% Selected by Respondents
1.	Soil Conditions	68
2.	Site Access	46
3.	Traffic	39
4.	Project Details	32
5.	Client Specifications	32

A similar list of the top five most significant factors for open cut is shown in Table 9. The differences between the decision factors for open cut and HDD are location of existing utilities and the impact restoration. Existing utilities are a major concern for traditional open cut because once the utility location is identified it must be exposed and crossed. For HDD, after the location of the existing utility has been identified, any contact with the utility can be avoided through steering the drill head. Sixty-one per cent of respondents deemed that soil conditions were the most important factor for open cut construction and 35% of these specifically indicated ground water table as being their issue with soil conditions. Existing utilities is notably the most surprising because in section five of the survey, contractors identified existing utilities as being more important for HDD than open cut construction.

Table 9. Contractors' Top Five Factors for Traditional Open Cut

Open Cut Factors	% Selected by Respondents
1. Soil Conditions	61
2. Traffic	46
3. Existing Utilities	39
4. Project Details	25
5. Restoration	21

Knowledge and Awareness Level

In the last section of the survey, contractors were asked to rate the knowledge level of engineers and owners involved in typical underground construction. From those surveyed, the knowledge level for engineers was 4.8 out of 10. We were not surprised by this low score because of the lack of education in underground construction. The average knowledge level for an owner was 3.9 out of 10.

CONCLUSIONS AND RECOMMENDATIONS

Selection of the most feasible construction method is becoming increasingly more important as municipalities and their infrastructure develop. Construction solutions are usually solved with the lowest cost method; however, with an increase in public concerns regarding environmental issues, other factors should also be considered. Eco-social effects, construction consideration factors and risk factors have become critical in selecting the preferred construction method.

Surveys were distributed to contractors with a broad range of annual revenues across the United States and Canada. From the surveys gathered, an analysis was performed to compare types of risk factors and costs found in HDD and open cut construction. According to the results gathered from section three of the survey, previous experience is the most important factor in estimating a contingency plan. The contractors indicated that the most important factor affecting the bid price of a project was the type of construction method, followed by the contractor's experience and the quality of the cost estimate. In terms of a direct comparison of the two underground construction methods, two-thirds of the factors studied had a savings for HDD. The greatest savings for HDD were the restoration costs, while the greatest savings for open cut was equipment operation costs. The greatest comparable difference existing between HDD and open cut are dust pollution and traffic restrictions. The survey asked contractors for the top five influencing factors on a HDD and open cut project. In both lists, the top factor cited was soil conditions.

Results of the survey suggest that using HDD in urban environments not only has an economical advantage but also a reduction in risk factors that contribute to a successful project. Subsequently, owners that perform a comprehensive analysis on their projects will be able to determine the best construction method that will meet the owner's function, budget and safety needs.

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REFERENCES

Allouche, E.N., Ariaratnam, S.T., and Lueke, J.S. (2000). "Horizontal Directional Drilling: A Profile of an Emerging Underground Construction Industry", *Journal of Construction Engineering and Management*, Vol. 126, No. 1, pp. 68-76.

Ariaratnam, S.T., Lueke, J.S., and Anderson, E (2004). "Reducing Risks in Unfavorable Ground Conditions during Horizontal Directional Drilling", *Practice Periodical on Structural Design and Construction*, Vol. 9, No.3, pp. 164-169.

Barras, G and Mayo, R (1995). "Minidirectional Drilling for Installation of Underground Electrical Conduit", *Journal of Construction Engineering and Management*, Vol. 121, No. 4, pp. 364-369.

Knight, M., Duyvestyn, G., and Gelinas, M. (2001). "Excavation of Surface Installed Pipeline", *Journal of Infrastructure System*, Vol. 7, No. 3, pp.123-129.

Kramer, S.R. (1992). *An Introduction to Trenchless Technology*. MacDonald, W.J and Thomson, J.C. (editors). Chapman and Hall, New York, NY, pp 176-181.

Lueke, J.S. and Ariaratnam, S.T. (2005). "Surface Heave Mechanisms in Horizontal Directional Drilling", *Journal of Construction Engineering and Management*, Vol. 131, No. 5, pp. 540-547.

SEYMOUR-CAPILANO WATER FILTRATION PROJECT STEEL - THE PRODUCT OF CHOICE

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1.0 ABSTRACT

The Greater Vancouver Water District (GVWD) is constructing the $600 million Seymour-Capilano water filtration project to enhance drinking water quality in the Vancouver metropolitan area. The project comprises the Seymour-Capilano Filtration Plant, the Capilano Pumping Station, the Twin Tunnels with associated shafts and the Capilano Break Head Tank/Energy Recovery Facility. The Twin Tunnels will convey untreated, raw water under pressure from the Capilano Reservoir to the Seymour-Capilano Filtration Plant and return treated water for distribution to the member municipalities. The tunnel system comprises twin 3.7 meter diameter tunnels approximately 7.2 km in length, the 11 meter diameter, 180 meter deep Seymour shaft as the main launch shaft, and two 4 meter diameter, 268 meter deep raisebore exit shafts at Capilano.

Among the numerous complexities of a project of this scope was the selection of pipe materials, including the lined Twin Tunnels portion of the project. Steel pipe was the only tunnel lining material, of several that were considered, that offered the required combination of being completely water tight at the high water pressures associated with this application, capable of resisting the buckling loads, providing seismic properties required and providing acceptable constructability. The GVWD also has a long history and good experience of using steel pipe as the transmission water main material of choice. These items and high seismic design standard led the GVWD to select steel pipe for use on the Seymour-Capilano filtration project.

Project construction commenced in 2003 and is expected to be complete by the end of 2008. Once completed, the GVWD will reliably deliver water of quality in full

compliance with the Canadian Drinking Water Quality Guidelines and the Provincial Drinking Water Protection Regulation.

2.0 INTRODUCTION

The GVWD is a wholesale drinking water supplier, providing water to approximately 2 million people in 18 municipalities in the Lower Mainland, British Columbia, Canada. The total land area of the system is over 2000 square kilometers, excluding the watersheds.

The water is collected from the rainwater and snowmelt that falls on the mountains of the Capilano, Seymour and Coquitlam watersheds, a 585 square kilometer area where access is strictly controlled in order to safeguard the quality and security of the supply. The water is held in six mountain storage lakes and distributed to communities through a network of dams, pumping stations, service reservoirs, and transmission mains, as shown in Figure 1.

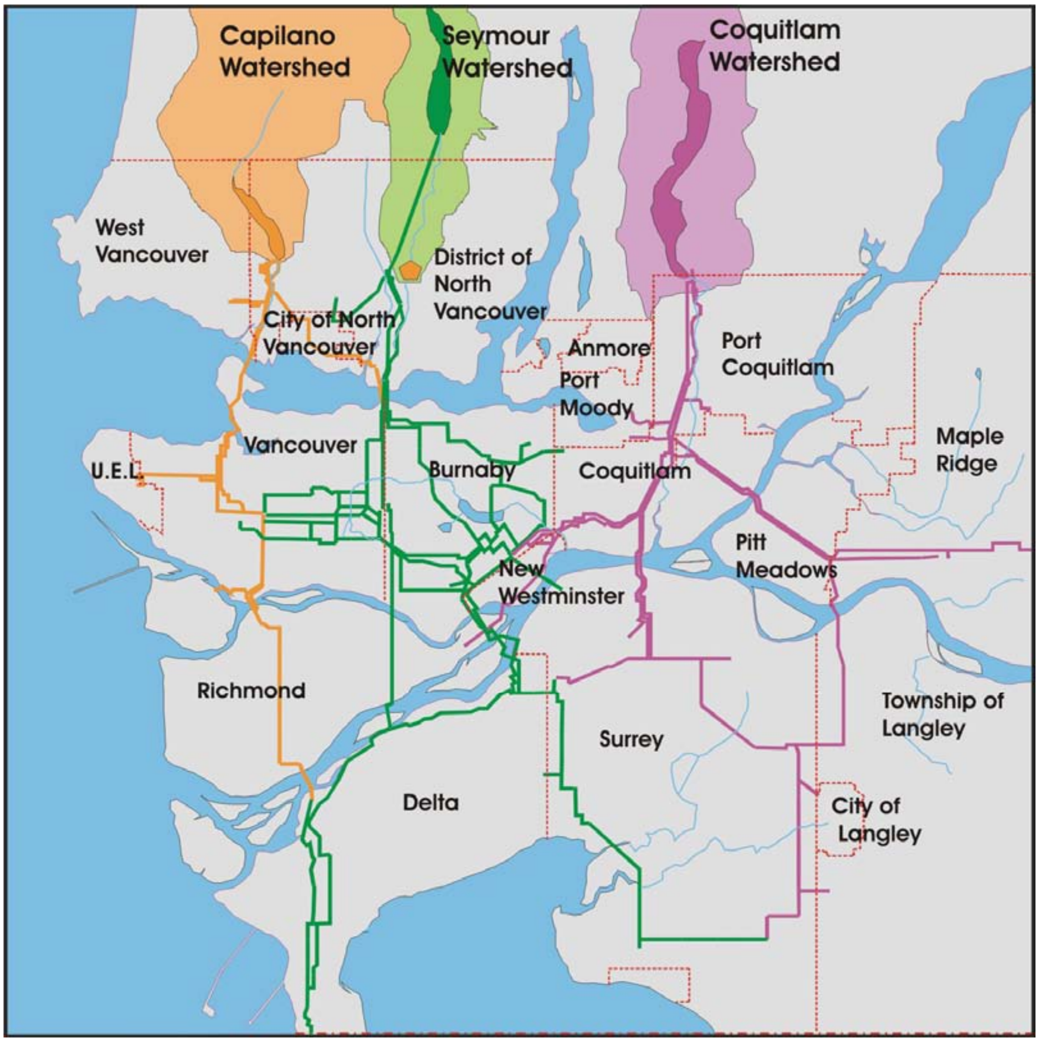


Figure 1. GVWD distribution system

The GVWD is a partnership of municipalities, and as such, the regional water system is in effect owned by the member municipalities. It is operated on a not-for-profit basis, with water delivered to all members at the same rate of 34.80 cents per 1,000 litres (2007 rate). The members, in turn, are responsible for distribution to consumers within their borders. The rate charged to consumers (residential, commercial, industrial, etc.) is determined by each municipality.

An overview of the GVWD and municipal water system is presented in Table 1.

Table 1 - The System at a Glance

First water main constructed	1889
Population served	2 million
Municipalities served	18
Distribution system, land area	Over 2,000 square kilometres
Watershed area	585 square kilometers
Dams	6
Storage lakes	6
GVWD reservoirs	22
GVWD pumping stations	15
Supply and transmission mains	Over 500 kilometres (300 mi)
Municipal distribution mains	Over 7,500 kilometres (4660 mi)
Average daily consumption (2004)	1.2 billion litres (317 million gallons)
Record one-day consumption (July 30, 1990)	2 billion litres (528 million gallons)
Cost of water to municipalities (2007)	34.80 cents per 1,000 litres (.13 cents per gallon)
Approximate average municipal water rate (2007)	$160 per household per year
Value of GVWD assets	Over $2 billion

The GVWD has a long history of design and construction of steel pipelines. The existing GVWD water main pipe summary is shown in Table 2. This summary confirms the GVWD preference to use steel pipe in its systems. Over the years, a wealth of practical design and construction experience has been developed. In the early 1960’s, the original design engineering standard was established to provide a reference for GVWD pipeline designers.

Table 2 - GVWD Water Mains – Pipe Length (km)

Pipe \ Age	0-24 years	25-49 years	50-74 years	75-99 years	>100 years	Total	Size mm (inch)
AC		3				3	300-1050mm (12-42 inch)
Cast Iron			1			1	<450 mm (<18 inch)
Ductile Iron	3					3	600 mm (24 inch)
PVC	1	1				2	<400 mm (<16 inch)
Steel	137	194	93	32		456	600 -2400 mm (24-96 inch)
Stainless Steel	1					1	3000 mm (120 inch)
PCCP & Cast in Place concrete	20	2	7			29	1000-1650 mm (39-66 inch)
Other	3	3				6	<300 mm (<12 inch)
System static pressure varies 700-2300 kPa (100-330 psi)						Approx. 500 km	

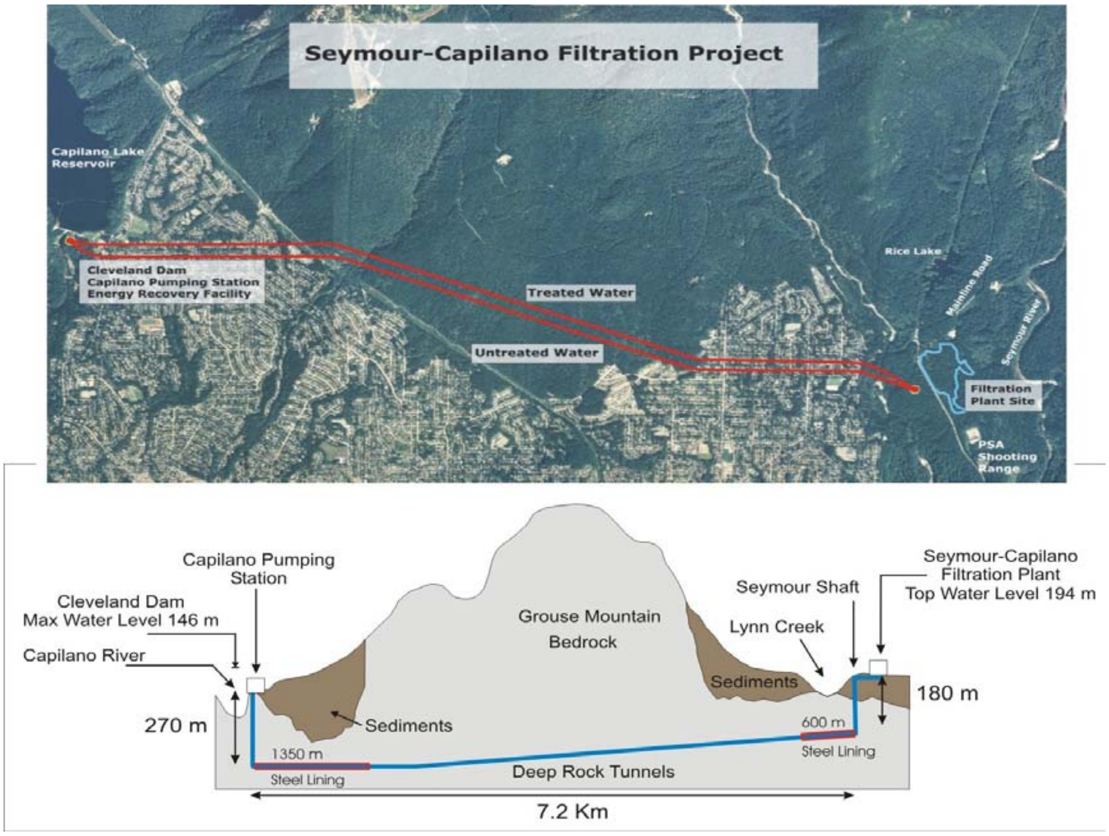
3.0 FILTRATION PROJECT COMPONENTS

Greater Vancouver's water system – which had its beginnings in the late 1880s as a single main from the Capilano River to downtown Vancouver – has grown to become a network of highly complex infrastructure.

An extensive program is currently underway to upgrade water supply and treatment facilities over the next decade to meet water quality standards and the needs of an expanding population. Waters from the Capilano and Seymour sources provide about 70% of the region's drinking water.

Filtration of the Seymour and Capilano water sources will provide significant benefits to the region. In addition to improving drinking water by removing microorganisms, organics, and silts and clays caused by heavy rainfall, filtration and UV primary disinfection reduces the amount of chlorine required to maintain water quality.

The Seymour-Capilano Filtration Project, which will filter up to 1,800 million litres of water per day from the Seymour and Capilano sources, includes the following facilities:



3.1 Seymour-Capilano Filtration Plant

The plant, with UV disinfection and clearwells (reservoirs), will be located in the Lower Seymour Conservation Reserve, a natural area south of the Seymour Falls Dam.

The facility will treat raw water through the processes of flocculation, direct filtration, UV primary disinfection, chlorine secondary disinfection and corrosion control. It is expected to be the largest filtration plant in Canada and the largest UV disinfection plant in North America until the planned New York City plant is operational in 2010.

3.2 Capilano Pumping Station

The 16,000 hp Capilano Pumping Station is located in the Capilano River Regional Park immediately below the Cleveland Dam and east of the Capilano River. The station will pump Capilano water to the filtration plant site through one of the tunnels.

3.3 Twin Tunnels

Twin Tunnels, each 7.2 km long and approximately 3.8 m in diameter, will be located approximately 130-640 m underground in bedrock and will convey water between the Seymour and Capilano sites. One tunnel will deliver raw water from the Capilano

Reservoir via the pumping station to the filtration plant for treatment, and the second tunnel will convey the treated water by gravity from the plant back to the Capilano distribution area. The tunnels are being constructed by two tunnel boring machines and will pass beneath the slopes of Grouse Mountain and Mount Fromme.

The quality of the rock that the tunnels will be bored through is expected to be sound, such that only a portion (up to 2 km in each tunnel) is expected to require steel lining. The tunnels are being constructed from a single shaft on the Seymour end, with individual pipe risers connecting each of the tunnels to surface pipelines to either the filtration plant inlet or outlet of the clearwell. At the Capilano end, separate shafts will connect each tunnel to the surface piping that leads either from the pumping station or to the Energy Recovery Facility/Bread Head Tank.

3.4 Energy Recovery Facility/Break Head Tank

A 2 MW Energy Recovery Facility, to be located near the Capilano River Regional Park public parking lot, will convert excess water pressure from the return tunnel water into electrical energy for load displacement or re-introduction into the BC Hydro grid.

The break head tank, a turbine tail and pressure reducing valve water tank downstream, of the energy recovery facility, will maintain a constant (improved) pressure on the Capilano distribution system. Pipelines will connect the treated water tunnel to the energy recovery facility and break head tank, and then to the Capilano distribution piping.

4.0 STEEL PIPE – WELDING AND JOINT DESIGN

Since the early 1960s, design standards have evolved and it has become apparent that more conservative design is necessary for certain applications. For the majority of applications, the GVRD continues to tender for bell and spigot joints and completes installation through a double joint design with great success and a proven history. Butt joints are typically considered by the GVRD for those projects that have a high degree of importance/cost or are subject to extreme conditions such as marine crossings, inaccessible tunnels, and low strength soil conditions where a high degree of settlement may be expected, etc.

Issues associated with bell and spigot pipe joints fabricated prior to 1986 were first encountered at highly difficult and extreme conditions installations, specifically marine crossings. In 1986, the American Waterworks Association (AWWA) standard for steel pipe was upgraded to require bell transition radii that minimize inherent stresses. While the butt joint represents the best possible/practical joint for highly difficult water transmission applications such as river crossings, the changes associated with the AWWA standard in 1986 for bell and spigot joints has made this type of joint the most common in steel pipe installations and has a proven history.

The GVRD developed a construction method where pipe was assembled on shore and pulled through a trench dredged across the water course. All of the bell and spigot pipe joints were double welded (i.e., welded both on the inside and outside of the pipe). This double welded design was originally thought to produce a joint stronger than the pipe wall. However, ruptures at the curve in the bell during the installation and initial pressure testing of some river crossings constructed prior to 1986, indicated that the processes to manufacture the bells created additional stresses in this portion of pipe. This type of problem in part initiated the bell and spigot bell transition radii related changes to AWWA C200 and increased designers awareness and opportunity to switch to butt-welded pipe joints for most marine crossings. More recently, problems began to occur in soft soil areas of land based pipelines where significant settlements (i.e., settlements in the order of metres) resulting from construction activities of others caused failures at bell and spigot joints. Due to these problems, GVRD began re-considering the design criteria for pipe joints as well as the specified steel yield strength in these extreme conditions. Consideration of the capacity of bell and spigot joints was carried out through a number of initiatives during the mid to late 1990's. Initiatives included literature review, attendance at various conferences, discussion with other agencies, review of pipeline design codes and finite element modeling and physical testing. Revised general guidelines for the design of new facilities and upgrading of existing infrastructure are summarized in Table 3. It is suggested that designers use these recommendations as a guideline for welded steel pipeline design. Users are to recognize that there will be situations where the recommended solutions will not be appropriate and professional judgment will be necessary to develop the design.

Table 3 – Welding and Joint Design Guideline

Item	Description	Recommendations
1.	Tube turn bends	Up to 24 inch dia. – YES > 24 inch dia. – No (use only if practical/required)
2.	Fabricated max. mitre bend angle per cut	22.5° (horizontal and combined) 11.25° (vertical)
3.	Pre-fabricated bends	Not required. Use of field fabricated bends is acceptable.
4.	Pipe wall thickness	Determine using GVRD Engineering Standards, "Steel Pipe Calculations, Pipe Wall Thickness".
	a) mitres	Potential movement area subject to further investigation. Increase F.S. in hoop stress formula from 2.0 to 2.5 at Engineer's discretion.

	b) bell and spigot joints	Do not increase wall thickness in firm ground. See item 5 for recommendations for soft soil.
5.	Geotechnical Conditions – soft soil conditions or conditions where significant movement can be anticipated	Avoid poor ground conditions where possible during route selection, improve poor ground where feasible and cost effective, increase pipe wall thickness (by using ASME Section VIII Division Joint efficiency factors) or change joint type (from bell and spigot to butt welds) or strengthen the pipeline by increasing yield strength.
6.	a) Welding	Qualify welding procedure (WPS/PQR) and welder operator. Testing and welding defects/repair to CSA Z662 (field) and to CSA Z245 (shop).
	b) NDT	Radiograph 10% of field mitres in accordance with AWWA C206 plus all mitres at major road crossings. Radiograph 100% of butt welded joints.
7.	Pipe Hatches	GVRD "boiler" type hatch.
8.	Pulled Joints	$\tan \varnothing = \frac{1.5 \text{ inch}}{D}$ (Min. 1.5 inch from the cold worked transition in the bell, min. 1 inch overlap or 3 x t whichever is greater) (based on 4 inch flat bell)
9.	Skewed Joints	Avoid Skew Joints. Use mitre cuts.
10.	Bell and Spigot Tolerances	5/64 inch (2 mm) all around max., gap, equally spaced. 2/64 inch (0.8 mm) all around min., gap, equally spaced.

5.0 SEISMIC DESIGN CRITERIA

The Greater Vancouver Regional District owns, operates and maintains the large-scale systems for storing, treating, and supplying potable water and for collecting, conveying, and treating wastewater and as such, it provides an essential service. These systems are expected to survive a moderate earthquake with minimal disruption and a severe earthquake with manageable disruptions. GVRD standardized the seismic criteria to which all new and remedial works are to be designed. The Seismic Design Criteria have been established based on the District's post disaster operating objectives and review of the public safety consequences of failure for each facility.

The seismic design criteria are provided as a guide for the designer and are to be considered as desired levels of seismic resistance rather than absolute minimum requirements. When designing new facilities or upgrading existing facilities, the cost of meeting the seismic design criteria must be weighed against the importance of the facility for system operation (i.e. redundancy in the system), and any life safety issues associated with damage to the facility. The table below presents the facility class and the expected performance levels for the NBCC and MCE levels of earthquake.

FACILITY	**NBCC 1:2475 Return Period**	**MCE 1:10000 Return Period**
Dams	IO	LS
Water Treatment Plants	IO	LS
Wastewater Treatment Plants	LS	-
Primary Water Disinfection Facilities	IO	LS
Secondary Water Disinfection Facilities	LS	-
Pump Stations – Level 1	IO	LS
Pump Stations – Level 2	LS	-
Pipelines(1)– Level 1	**IO**	**LS**
Pipelines(1) – Level 2	**LS**	-
Segmental Pipe (2)	LS	-
Chambers and Portals - Level 1	IO	LS
Chambers and Portals – Level 2	LS	-
Reservoirs – Level 1	IO	LS
Reservoirs – Level 2	LS	-
Ancillary Structures	CP	-

1) High pressure continuously bonded pipes such as welded steel, polyethylene or flanged steel.
2) Consult with pipe manufacturer on the recommended allowable deflection at the joint.

NBCC The National Building Code of Canada provides an Earthquake spectrum, which can be used for analytical modeling.

MCE The "Maximum Credible Earthquake" is site specific. The MCE design level is only appropriate for those facilities that present a high hazard to properties and life.

Level 1 Facility is critical to system operations, has little or no redundancy, and may result in substantially reduced service for an extended period (months) if failure occurs. This level includes pipelines for marine crossings.

Level 2 Facility is important to the system operations, however some level of redundancy exists, service will only be marginally impacted for a limited period (days to weeks) if failure occurs.

IO Immediate Occupancy, no damage

LS Life Safety, minor damage, operational at full capacity, repair duration 2 months

CP Collapse Prevention, moderate damage, operational at less than full capacity, repair duration 6 months

The summary table below gives the applicable category, according to the GVRD Seismic Design Criteria Engineering Standard, for each for the major project areas.

Project Element	Level	NBCC 1:2475	MCE 1:10,000
Filtration Plant	n/a	IO	LS
Capilano Pumping Station	1	IO	LS
Pipeline	1	IO	LS
Energy Recovery Facility/Break Head Tank	1	IO	LS
Twin Tunnels	1	IO	LS

6.0 STEEL THE PRODUCT OF CHOICE

6.1 <u>Design Considerations</u>

- Geotechnical
- Tunnel diameter
- Pipe diameter
- Internal pressure
- External pressure
- Cost
- Water quality
- Infiltration
- Exfiltration
- Raw water/treated water tunnel cross flow
- Future dewatering
- Seismic
- Social
- Environmental

Construction Considerations
- Pipe wall thickness and weight
- Pipe lengths
- Joint connection
- Corrosion protection
- Needs for mitering of tunnel liner
- Fittings and specials

Operation/Maintenance Considerations
- Reliability
- Consequences of failure
- The ease and convenience of operation and maintenance

6.2 Surface Pipe Selection

From evaluation of the items identified to be significant to the GVRD's needs, steel pipe meeting the requirements of AWWA C200 standard was chosen as the preferred pipe for this project. While all the considerations listed were assessed, the primary reasons for the GVRD to choose steel pipe were as follows:
- pipe diameter (2.7 m to 3.0 m)
- seismic performance
- long term service history for potable steel watermains in GVRD system
- design consultant and GVRD engineering/operation experience related to steel pipe

6.3 Tunnel Steel Linings Selections

The hydraulic grade line for both the raw and treated water tunnels is higher than the elevation of the surface topography at both ends of the tunnel alignment. These therefore represent sections of the twin tunnels where leakage may occur. Excessive leakage of both raw and treated water is undesirable and, in addition, such leakage could pressurize the overburden above the tunnels with various undesirable effects. The commonly adopted solution in pressure tunnel design for the prevention of leakage is the installation of steel linings. Reinforced concrete and membrane type lining solutions were reviewed and considered but were not determined as a preferred lining for these small diameter tunnels based on considerations noted above.

The design length of steel linings at both ends of the tunnel alignment has been based on consideration of preventing hydraulic jacking and preventing cross flow between the tunnels at their terminus. Since the hydro-jacking testing results indicated stress levels equivalent to the overburden, the concern was primarily of reducing small expected leakage during operations to control the pore water pressure and insure the surface slope stability.

Design of the steel liner closely followed ASCE 79 wherever possible. Liners in the shafts and tunnels were sized for the governing condition of a future dewatering event, i.e. external pressure on an empty pipeline, which primarily drove liner thickness. Although internal pressure did not specifically factor into thickness selection, hoop stresses for a design internal pressure of static head plus surge were checked to ensure values were within the allowable limits prescribed in ASCE 79.

Allowable hoop stresses were evaluated using criteria found in ASCE 79 and AWWA M11, with ASCE 79 being the more stringent of the two codes. The shaft and tunnel steel liner will be 3 m diameter with the pipe wall thickness between 12 mm and 34 mm.

a) Seymour Section

The maximum differential outward pressure at the Seymour (eastern) end of the tunnel alignment is about 70 m of water. Lining is therefore required along this section to prevent leakage and impacts to any existing surface structures/slopes. The length of steel lining required at the Seymour end of the alignment is approximately 600 m. This has been based on extending the steel lining beneath Lynn Canyon, where a major fault/shear is inferred to be present, to beyond the crest of a natural overburden slope along which residential housing is located. The steel lining along this section of the tunnel alignment has been designed for full hydrostatic loading of about 150 m of water upon dewatering of the tunnels.

b) Capilano Section

The maximum differential outward pressure at the Capilano (western) end of the tunnel alignment is about 60 m of water and therefore lining is also required along this section, both to prevent leakage and to prevent any possible effects on the east abutment of the Cleveland Dam. At the Capilano end of the tunnel alignment the length of steel linings required is approximately 1200 m. This has been based on extending the steel linings eastwards to the location where the hydraulic grade line intersects the groundwater table, beyond which leakage is theoretically impossible. A key requirement is that the tunnels must have no effect on the groundwater pressures in the east abutment of the Cleveland Dam. This length of steel lining extends eastwards across a major inferred fault/shear zone associated with Capilano buried glacial valley. The length and cost of the installation of steel lining at the Capilano end of the tunnel alignment is significant to the Twin Tunnels Project. The steel lining along this section of the tunnel alignment has been designed for full hydrostatic loading of about 305 m of groundwater upon dewatering of the tunnels.

c) Middle Sections

The underground water table in the middle section is higher than the operating tunnel hydraulic grade line, the small leakage is not a concern and the tunnel liner is not required for this purpose.

Also, the potential for metals/minerals to leach into the drinking water supply from the exposed rock in the unlined tunnel section was reviewed. The following is the summary of our review:

1. Preliminary results of exploratory drilling from the ground surface in 2004 before tunnel construction began indicated that there was little or no potential for minerals to leach into water sampled from test bore holes.
2. To date only the Seymour shaft section of rock has been excavated and tested. Testing of the shaft rock to date for leachability potential into water has found no metal concentrations of health concern when compared with the Guidelines for Canadian Drinking Water Quality.
3. As the tunnel boring continues, the excavated rock will be tested on a regular basis, such that where tunnel rock walls are exposed to drinking water (approximately 5 km per tunnel), the potential for leaching of metals/minerals into the drinking water will be determined.
4. Based on the shaft rock results, the potential for leaching of metal/minerals from the unlined tunnel section is believed to be very low (i.e. negligible).

Considering the above noted, the tunnel liner in the middle section will most likely not be required.

d) <u>Shafts</u>

On completion of the steel lining in the tunnels, the 3 m diameter Capilano shafts will be steel lined throughout their depth. Towards the end of construction, the Seymour shaft will be backfilled and, at the same time, two steel riser pipes will be installed within the shaft. At both the Seymour and Capilano ends of the Twin Tunnels short surface pipelines will connect the shaft risers to other surface facilities.

7.0 CONCLUSIONS

Based on the foregoing, the following conclusions/best practices are drawn:

i) Applicable standards and a design criteria need to be established prior to starting detailed design.
ii) Development and use of the welding and joint design criteria should be done to suit the intended project application.
iii) Steel pipe works well in large diameter, high internal pressure, high external pressure and difficult installation pipe applications.

References:
1) *GVRD Engineering Standards*
   - *Seismic design consideration*
   - *Steel pipe – welding and joint design*

2) *Seymour Capilano Filtration: D. Neden, M. Ferguson, G. Oljaca – Innovation March 2003*
3) *The Seymour Capilano Twin Tunnels Project, D. Brox, B. Garrod, T. Morrison, A. Saltis*

ASCE Pipelines 2007

Lake Tawakoni Water Supply Project, Critical Path Issues and Lessons Learned: Fast-Tracking a $100 million Water Transmission Project

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Abstract

Since its formation in 1951, the North Texas Municipal Water District has been charged with developing a reliable water supply source for its 13 member cities and 46 other customers (some direct and some indirect), including more than 1.5 million people in portions of north Texas including portions of Collin, Dallas, Denton, and Kaufman Counties and all of Rockwall County. Recent long-range water supply planning efforts have identified significant increases in water demands that must be met through a corresponding increase in available raw water supply. Compounding the need for raw water is the fact that the North Texas region is currently in the second year of a drought that is the worst on record since the 1950s, with the District's primary water supply reservoir, Lake Lavon, falling some 17 feet in water surface elevation since mid-2005.

In an effort to supplement available supplies, the District contracted for 50,000 acre-feet of water per year from the Sabine River Authority in Lake Tawakoni, located approximately 45 miles east of Dallas in October 2005. Given current drought conditions and ominous rainfall forecasts, the District has requested that the infrastructure needed to transfer this water from Lake Tawakoni to Lake Lavon be in service no later than October 1, 2007, to prevent further water restrictions and possible water rationing. The infrastructure needed includes approximately 30 miles of 54 and 60-inch-diameter transmission pipeline and two pump stations, each with a raw water pumping capacity of 75 MGD. The estimated project cost is $100 million. Critical path issues for meeting the highly accelerated schedule include overall project team coordination; route selection; environmental permitting considerations; easement and property acquisition; power supply; equipment delivery; and construction of multiple project components simultaneously. The focus of this paper is to discuss the critical path issues and offer lessons learned from fast-tracking the design and construction of a multi-million dollar water transmission project.

Introduction

The North Texas Municipal Water District (NTMWD) is a water supply and reclamation district created by an act of the Texas Legislature in 1951. The District generally serves a five county, 1,975 square-mile area encompassing portions of Collin, Dallas, Denton and Kaufman Counties, and all of Rockwall County. The District provides treated water to 13 member cities and 46 other customers (some direct and some indirect), which have a combined population of more than 1.5 million people. The District supplies wholesale treated water to one of the fastest growing areas in the United States. Some cities in the District's service area have experienced nearly 10% compounded growth rates in the last 10 years.

The District currently obtains its raw water supply from Lake Lavon, Lake Texoma, Lake Chapman, and reuse of treated wastewater effluent from the Wilson Creek Regional Wastewater Treatment Plant. Currently, the District has approximately 274,000 acre-feet of permitted water rights from all sources combined. Raw water from Lake Texoma is diverted via a 90 million gallons per day (MGD) pump station and 25 miles of 72-inch pipe to the West Prong of Sister Grove Creek near Howe, Texas. Once discharged into the West Prong of Sister Grove Creek, the water diverted from Lake Texoma then flows by gravity to Lake Lavon. Raw water from Lake Chapman is diverted via a 110 MGD pump station on Lake Chapman and pumped through 39 miles of 84-inch pipeline to Hickory Creek, where Lake Chapman water flows into the Pilot Grove Creek arm of Lake Lavon. Treated effluent from Wilson Creek WWTP is discharged into the East Fork arm of Lake Lavon. The District already has gone to great effort to move significant volumes of raw water to Lake Lavon from surrounding reservoirs. The District is constantly and continuously exploring all available avenues and options to obtain additional water supplies to meet their ever growing demands.

Project Need / Project Description

In an effort to supplement available supplies, the District contracted for 50,000 acre-feet of water per year in October 2005 from the Sabine River Authority in Lake Tawakoni, located approximately 45 miles east of Dallas. Given current drought conditions and ominous rainfall forecasts, the District has requested that the infrastructure needed to transfer this water from Lake Tawakoni to Lake Lavon be in service no later than October 1, 2007, to prevent further water restrictions and possible water rationing. Since mid-2005, Lake Lavon, the District's primary raw water supply, has dropped 17 feet in elevation. Rain in early 2007 has produced an rise in the lake elevation as of March 2007; however, the lake remains more than seven feet below normal pool. The infrastructure needed includes approximately 30 miles of 54 and 60-inch-diameter raw water transmission pipeline and two pump stations, each with a raw water pumping capacity of 75 MGD. As shown in Figure 1, these facilities will connect to the District's East Fork Raw Water Conveyance Pipeline, which is currently under construction to convey raw water from a constructed wetland near Combine, Texas, to Lake Lavon.

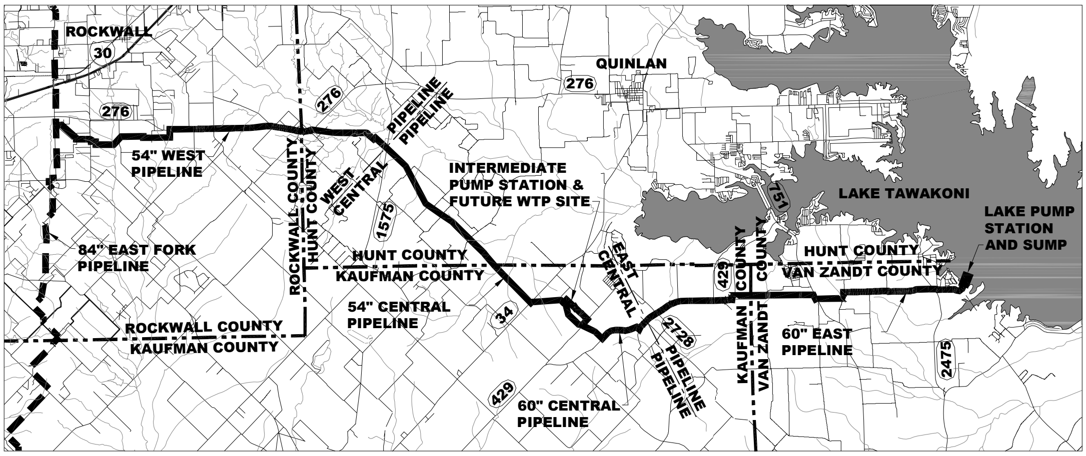

Figure 1. Overall Project Area

Preliminary Project Planning

Without question, the key factor in the team's ability to deliver a $100 million project in less than 10 months is the trusted advisor relationship that has been formed over the last 50 years between the District and Freese and Nichols. Our intimate knowledge of the District's raw and treated water systems allowed us to move quickly once the contract for new water had been signed, beginning with an evaluation of options for incorporating the new water into the existing system. Timely evaluation of alternatives was critical, in that each week spent evaluating options was a week lost on the path to delivering the water to the District's system.

Over a six-week period, the team evaluated options for delivering the raw water to Lake Lavon (the District's primary water-supply reservoir) and options for treating the water at the existing plant in Wylie or at a possible new treatment plant in their South System. These options were analyzed using the District's existing system models and capital improvements plan, both of which Freese and Nichols has been involved in updating over the years. Without that prior knowledge, and more importantly, without the excellent working relationship with the District staff, the planning phase of the project could have taken significantly longer than six weeks.

The culmination of the planning phase of the project was the facilities described above and shown in Figure 1. An interesting feature of the project is the phasing plan, which includes construction of the 30-mile pipeline and the Lake and Intermediate Pump Stations all for the transfer of raw water initially. These will be on-line in the fall of 2007. By 2010, the South Water Treatment Plant will be constructed and a new potable water transmission line built to tie the new plant to the District's South System. At this point, the Intermediate Pump Station will be converted such that half of the capacity (35 to 40 MGD) will be treated water pumped to the South System and the remaining capacity will be raw water pumped to Lake Lavon. By 2020, the 54" section of the pipeline will be converted to treated water delivering water to the District's South and East Systems and the remaining Intermediate Pump Station capacity converted to treated water.

Project Management and Organization

In order to obtain a quality design given the accelerated project schedule, the team focused on five critical areas in organizing the project. These areas included construction contract organization, team organization, project management practices, project communications, and owner-specific actions.

The first key decision for the team was to determine the organization of the construction contracts. To meet a design and construction schedule of 22 months (10 months for design and 12 months for construction) for a project of this scope, the project was divided into multiple bid packages. A natural separation would be to divide the project into three separate construction contracts, one for each pump station and one for the pipeline. However, given long lead times for equipment and limited time for construction the project was ultimately designed and bid in seven different contracts. Each of these is listed below with a brief discussion of the impacts to project design and construction time.

- <u>Contract 1 – Substation Electrical Equipment:</u> This contract was originally included to facilitate timely procurement of the large substation transformers required at each pump station site. The lead times on this equipment were quoted at more than 12 months during early conversations with suppliers. Ultimately this contract was removed from the project, as the District was able to coordinate temporary power at the sites to operate the pump stations until permanent power could be provided by the utilities.

- <u>Contract 2 – Pumps, Control Valves, and Variable Frequency Drives:</u> Discussions with equipment suppliers indicated lead times on this equipment at about 12 months. System hydraulics and preliminary design of the facilities was completed in about five months. Considering the 12 month delivery schedule, the equipment is scheduled to be on site in July of 2007. This will allow two to three months for installation to meet the October 2007 in-service date of the raw water supply system.

- <u>Contract 3 – Lake Pump Station Sump and Intake:</u> This contract includes the below-grade facilities at the Lake Tawakoni Pump Station, including the pump station intake and connection to the existing intake pipeline and construction of the pump station wet well. Design time for these facilities spanned about seven months. By separating this work from the overall pump station construction, bidding and construction could occur as soon as design was complete instead of waiting until the complete pump station facility was designed. This saved as much as three months in design time and allowed construction to begin in October 2006.

- <u>Contract 4 – Lake and Intermediate Pump Station:</u> Contract 4 includes the above-ground pump station facilities at the lake and the entire Intermediate Pump Station and ground storage tank construction. Several issues were considered when evaluating the feasibility of combining these two sites into a single contract. A single point of responsibility was desired so that completion of the stations could be managed more effectively, operation of the system would be simplified since all electronics and programming would be completed by the same entity, and the likelihood of problems during construction would be reduced since only large contractors would have the bonding capacity to bid the combined project. Design of these facilities was completed on a 10-month schedule, allowing 12 months for construction.

- Contract 5 – West Pipeline: The West Pipeline includes about 10 miles of the 54-inch-diameter pipeline. Pipeline contracts of not more than 10 miles were selected to keep construction schedules to about nine months. Additional consideration was given to delaying the bidding of the pipelines as long as possible to allow the District more time to acquire easements for the project. As such, the West Pipeline design and survey was completed in October 2006 to allow time for pipe procurement; but construction could not start until late February 2007 to allow time for easement acquisition.

- Contract 6 – Central Pipeline: The Central Pipeline included about 8 miles of 54-inch-diameter pipe and 2 miles of 60-inch-diameter pipe. Design and survey for this contract was completed in November 2006, with construction beginning in February 2007.

- Contract 7 – East Pipeline: The East Pipeline includes about 10 miles of 60-inch-diameter pipeline with design and survey for this contract completed in November 2006, and construction beginning in February 2007.

Once the individual contracts had been identified, the next hurdle was to organize a project team scattered over different offices to design the facilities included in the six contracts. The approach used centralized project management to ensure project consistency and quality but decentralized execution to expedite the design work and ensure schedules were met. Figure 2 provides the overall project team organizational chart.

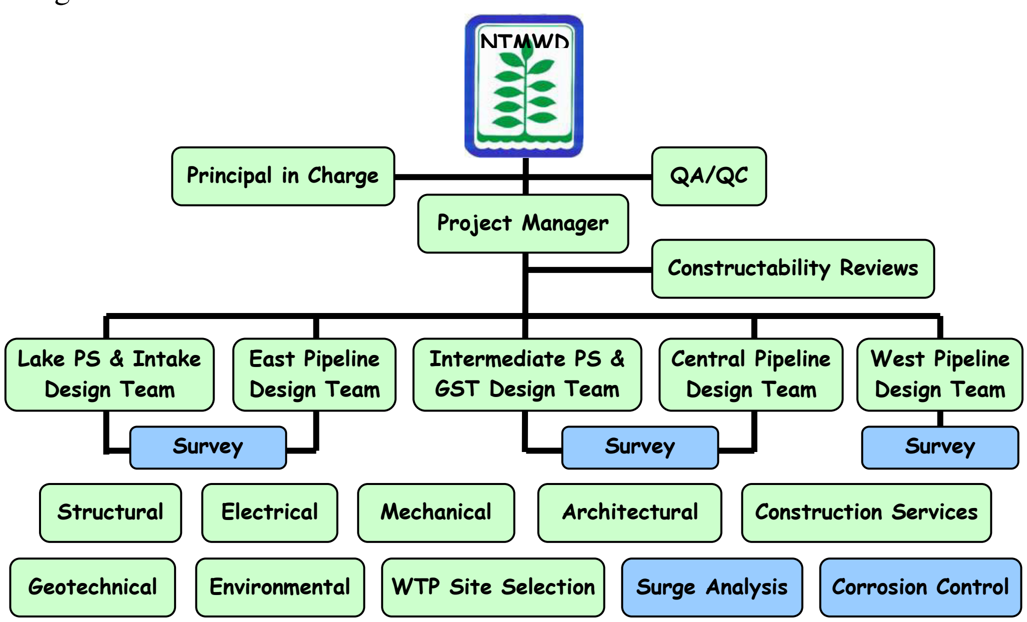

Figure 2. Project Team Organizational Chart

As shown in Figure 2, the project team included an overall project manager with phase managers for each of the six discrete construction contracts. The overall project manager served as the single point of contact between the District and the project teams to ensure all design teams were meeting the Owner's goals and by limiting the points of contact for the Owner. In addition, the project manager set common design standards, milestones, and internal quality control processes. Finally, the project manager organized the phase managers and ensured the phase teams had the resources necessary to complete the work.

The phase managers were responsible for the design of their respective facilities. Incorporating the direction and guidelines set by the project manager, these phase managers acted independently during the design process, but if issues that the phase managers could not address arose during the project, they were elevated to the project manager who provided direction or worked with the Owner to find a solution. Once an issue was elevated to the project manager, the project manager handling that item allowed the phase managers to continue to focus on the design task at hand.

In concert with the individual phase manager approach, the project team was organized to provide separate surveyors for each design phase. This ensured that each phase manager could independently work with a single surveyor and utilize that surveyor's resources only on their phase without having to compete with other phases for a surveyor's time. With a single surveyor assigned to each phase, the survey and easement preparation for all phases could proceed concurrently eliminating a significant amount of time from the required design schedule.

Because so much of any complex project is efficient and consistent computer aided drafting (CAD), the project team designated a lead CAD technician who was responsible for ensuring that applicable CAD standards were consistently applied across all design teams. The lead CAD technician served as clearing house for all CAD issues and worked to resolve issues across geographic locations as they arose.

All aspects of team organization and development would have been for naught if the individuals selected for the teams did not possess the characteristics needed for such a fast-paced project. The initial selection of the design team focused on an individual's ability to communicate, to be flexible during the fast-paced design, to work well within a large team, and the willingness to spend time in other offices as needed to coordinate and resolve issues in person. These individual characteristics were critical for team cohesion and success. Individuals who did not demonstrate these characteristics were not selected to participate on this project.

Once the team structure and composition was established, internal management processes were established. The first and most important of these was work sharing between multiple offices. In many instances, one office lacked the resources such as engineering or CAD to successfully complete the design. These gaps were identified and quickly filled by matching needs with personnel in other offices. For example, the engineering design of the Central Pipeline segment was performed in the Freese and Nichols Dallas office, but the CAD was performed by a technician in the Freese and Nichols Fort Worth office that had the availability and the individual characteristics to be a successful member of the team. In this situation it was important for the engineers to be willing to travel to the Fort Worth office periodically to discuss project issues with the CAD member.

Next, because certain design items and tasks were common to all or many of the design phases, the project manager was given specific responsibility for these items. These responsibilities included the standard details for the three pipeline segments, corrosion control design and surge analysis and control design for the project. Since the standard details were common to all three pipeline segments, the project manager was able to ensure consistency for all three contracts while eliminating the tripling of effort if all three pipeline phase teams were to develop and refine the details for their specific projects. The corrosion control and surge analysis was performed by specialty sub-consultants. Since the corrosion control design and surge analysis had to be applied holistically to the entire project, the project manager handled all coordination with these firms while utilizing information and designs the phase teams were providing. This approach again allowed the phase managers to focus on design issues unique to their segments and also allowed the sub-consultants to only deal with a single point of contact (i.e., the Project Manager), thereby eliminating the chances for confusion and inconsistencies between contracts.

Any project will fail without efficient and deliberate communications practices and procedures. When the project has a diverse project team and extremely aggressive schedule, efficient communications are even more paramount. The project team utilized multiple technologies to enhance project communications. The first of these was extensive use of video-teleconferencing between design offices. This enabled work sharing to be even more effective by allowing the geographically separated team members to conduct regular meetings and effectively address issues. Next, the team set-up an extensive folder organization on its file transfer protocol (FTP) site so that all parties could effectively share and transfer large documents without relying on individual email servers for the service, and the FTP site served as a central repository of design documents and information. Finally, the project team extensively utilized the calendar functions in Microsoft Outlook to jointly schedule project meetings. Both parties granted each other the ability to forward and schedule meetings on behalf of another user, which allowed greater flexibility, speed, and accuracy in scheduling project-critical meetings.

In addition to using technology to enhance communications, the project team also held weekly progress update and decision meetings. These meetings served as an avenue for the Owner and Engineer to discuss critical issues and keep important decisions in front of the group. These meetings were supplemented by monthly project update reports which consolidated all the design progress, action items, and decisions needed into a single, easy-to-read format the Owner the Engineer could utilize to track critical items and progress in all design phases. Finally, the project team developed a graphical means to track status of the acquisition of the more than 180 easements for the project. This format included a color-coded map that showed where each affected property parcel was in the acquisition process. This format was updated weekly and allowed the Owner and the engineers to quickly understand and report the process of right of way acquisition for the pipeline and pump station sites.

The Owner in any project plays a critical and often understated role. In its role, the District made critical decisions early in the project that were key to the team meeting the design goals and schedule. Most important was the continued top-down support and championing for the project among the District's staff. From the highest ranking officials inside the organization there was constant interest, support, and resources provided enabling their staff to keep the project on-schedule. One of the largest indications of this high-level support was the departure from the District's right-of-way acquisition processes. In the past, real estate acquisition was effectively handled by in-house staff. However, realizing that the large number of properties involved combined with the aggressive schedule was not manageable for current staffing levels, the District hired an administrative assistant dedicated to the project and contracted with an outside land acquisition firm to help meet the project schedule.

Design Issues

The first step in the design phase was the pipeline route selection. Tools used in route selection for this project included aerial images and contour data obtained from the United States Geological Survey (USGS), and county appraisal district maps. This information along with site visits was used to locate existing structures, roads, waterways, and utility corridors in the area. Several alternate routes were developed along with USGS contour data to assess the system hydraulics. The routes were evaluated based on pipeline length, capital cost, system hydraulics, and potential land issues. Midway through the route selection process when a general alignment had been selected, an aerial surveyor began flying the corridor to produce higher quality aerial images to help speed up the design process. Once the route was selected, land surveyors began surveying the route, researching property information, and preparing easement documents. During this process preliminary plans were developed with property information and USGS contour data to develop a rough profile. These plans included the selected alignment with a general profile and appurtenance locations that helped the land agents get a head start on easement acquisitions. In order to speed up the design process, a typical preliminary design report was not prepared. The preliminary plans were the only major early submittal from the pipeline design teams. A final design report will be submitted following the award of all of the construction contracts.

Selection of the future Water Treatment Plant (WTP) site was crucial in the selection of the 30-mile pipeline route. The site needed to be located on a large enough tract of land to accommodate the future WTP, the Intermediate Pump Station (IPS), and Ground Storage Tank (GST) while providing adequate access and topography suitable for a WTP. The site also needed to be located at a high enough elevation to optimize the hydraulics of the system. These factors were balanced along with an attempt to minimize the overall length of the pipeline. Several preliminary sites were selected based on the required criteria. Early on in the design process the District began negotiating with property owners of the potential WTP sites which were narrowed down early enough to finalize the pipeline route and facilitate design.

Early in the route selection the design teams considered the potential environmental permitting issues and impacts to the shortened project schedule. The decision was made early on to avoid impacts to any wetlands or other jurisdictional areas where possible and eliminate construction work in Lake Tawakoni in order to qualify for the non-notification section of the Nationwide 12 Permit. Processing of Nationwide 12 notification can take anywhere from 6 months to 2 years and could have proved to be a costly critical path item. Early during the route selection process, environmental and archaeological surveys were conducted along the pipeline corridor to designate sensitive areas. In order to avoid any construction in the lake, the intake pump station was tied into an existing intake pipe owned by the city of Terrell, Texas.

Early in the route selection process the design teams began coordination activities with governmental agencies and utility owners affected by the project. Information was gathered regarding the necessary permits and regulations to avoid potential conflicts along the alignment. In several cases the District was able to negotiate agreements with existing utility owners to share existing easements thus reducing the required easement width and impacts to land owners. The shared easements helped to simplify and speed up easement acquisition. Early coordination helped eliminate major design changes and allowed for early preparation of permit applications. Once the design was nearing completion the necessary permit applications were submitted in bundles to help speed up the approval process.

As with most pipeline projects, easement acquisition proved to be on the critical path. In order to minimize the duration of this task, surveyors worked to minimize the time period between deed research, easement preparation, and acquisition. With such a fast track schedule, the amount of property that switched hands between deed research and easement acquisition was minimized. This helped easement acquisition proceed with only minimal changes to easement documents. During the easement submittal and acquisition process the design teams developed and maintained detailed landowner information databases and coordinated maps with parcel ownership information that were color coded based on the status of each parcel. These tools were essential in keeping track of the status of the 180 easements to be acquired for the project. Since easement negotiations with landowners began during the design phase, it was essential that the design teams were flexible in making quick pipeline alignment changes and design adjustments. Several negotiations with land owners could have potentially delayed project schedule. To avoid project delays, work continued on other sections of the project while alignment decisions were pending. Towards the end of the project the design teams and surveyors worked together to quickly finish changed or delayed sections of the pipeline.

Power supply for the lake and intermediate pump stations was also a critical path item for the project. Each pump station is located in an area served by a different utility company. This required coordinate with two separate entities for power supply issues. The initial concept was to bring permanent power supply to each site from known power transmission lines and along the proposed pipeline routes where necessary. Early negotiations with the power supply companies revealed that permanent power could not be supplied to the sites by the in-service date. As a result temporary power via transmission lines and/or generators would be delivered to each site in anticipation of a permanent power supply source in the near future.

Construction Issues

During the planning and design stages of the project the team explored various methods to streamline and speed up the construction phase. Early discussions with equipment manufacturers and pipe suppliers allowed the design teams to properly plan and schedule bidding and construction to avoid major delays with material delivery. The pipeline was bid with material alternates to ensure a competitive price for the District and to avoid potential production line delays with pipe suppliers. Major pumps, valves, and control equipment was selected and bid with a 15-month lead time to avoid late equipment delivery.

The construction contracts were set up to avoid delays and optimize design production. The lake pump station work was split into two separate projects that allowed the excavation and lining of the sump pit to begin before design of pump station was completed. The lake and intermediate pump stations were combined into one construction contract to take advantage of standardization of SCADA, operation, control, setup. The combination of the two pump stations also streamlined the design effort requiring only one set of plans and specifications for both facilities. The construction contracts were also set up with the flexibility for alternate startup scenarios and the usage of temporary power supply if the permanent power source is not yet available.

Early on in the planning and design stages, the design team discussed potential construction issues with local contractors. The bonding capacity of contractors, competition in bidding, and construction incentives were all considered as potential methods to increase bidding interest and explore methods to accelerate the construction schedule. The construction contracts were split up within a range of construction cost to ensure that an adequate number of qualified contractors would be able to bid the projects. Contract specifications were set up to enable contractors to streamline the construction schedules as much as possible. It was also apparent during the design phase that all of the easements would not be acquired before the award of contracts. The pipeline specifications were set up to allow for sequenced land acquisition that would give the contractor access to different locations of the project by specific calendar dates. The specific construction contracts were discussed in previous sections of this paper.

The final component of the project was to provide construction phase services, including field representation for the project. Members of the design team were transitioned to the field in an effort to take advantage of their familiarity with the project design. This aids in completing the project quickly by reducing response time to contractors on submittals and questions in the field. The submittal review process has also been streamlined by using an entirely electronic web based format.

Lessons Learned

A project of this magnitude with an accelerated schedule resulted in many lessons learned that can be applied toward future projects. These include:

- Several problems with setting coordinate control and coordinate conversion arose from the use of 3 land surveyors and 1 aerial surveyor. Additional coordination at the beginning of the project on specific control issues could have prevented some of the issues.
- Problems came up with concerning existing utilities, easements, and existing land issues that caused several delays during design. Some of these issues could have been identified earlier to avoid changes to the design and delays to the project schedule.
- A project of this magnitude presented many opportunities for continued

learning and training of design team members during design. Some of these opportunities were not fully taken advantage of due to the accelerated project schedule.

- It was essential for the design teams to be flexible with making sudden design changes due to the accelerated project schedule.
- Good communication is vital for a fast paced project to have a successful outcome. The design teams generally maintained a good level of communication throughout the project, however there were a few instances where communication could have been improved
- Work sharing between offices was necessary with the size and compressed schedule of the project. Several portions of the project required engineers and CAD technicians from different offices to work together. Although the process was inherently slowed the down somewhat, across office coordination generally worked well. A few lessons that could be taken from this project would be increased visits by the engineer to other offices to increase face-to-face communication and the usage of color scanned markups of plans and specifications to avoid misunderstandings
- Picking the right people for the project team was one of the most essential aspects that made this a successful project. This fast paced project required a project team that was flexible, able to communicate and work with new project team members, and a willing to travel frequently between offices and put in extra hours when necessary.

Project Update

Construction for all phases of the Lake Tawakoni Water Supply Project is underway. Bid totals, key dates, and estimated costs for the construction contracts and all incidentals are listed in the Table 1 below:

Table 1. Bid Totals and Key Dates

Contract	Bid Date	Estimated Completion	Bid Amount
Contract No. 2, Equipment Pre-Selection	06/19/06	07/1/07	$ 4,180,000
Contract No. 3, Lake Sump & Intake	08/01/06	03/01/07	$ 2,850,000
Contract No. 4, Pump Stations	10/19/06	01/01/08	$ 16,680,000
Contract No. 5, West Pipeline	10/26/06	10/15/07	$ 17,330,000
Contract No. 6, Central Pipeline	11/15/06	10/15/07	$ 15,200,000
Contract No. 7, East Pipeline	11/02/06	10/15/07	$ 17,320,000
Subtotal Construction			**$ 73,560,000**
Total Power Supply			**$ 12,110,000**
Miscellaneous, Right-of-Way, and Professional Services			**$ 11,680,000**
Total Project Cost			**$ 97,350,000**

Nacimiento Water Project Intake Facility

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Abstract

Black & Veatch Corporation is directing a project team that is designing a new $185 million supplemental raw water supply from Lake Nacimiento for San Luis Obispo County. The project consists of a multi-port sloping intake facility and pump station, two intermediate pump stations, three storage tanks, control center, and approximately 45-miles of transmission pipeline ranging in diameter from 36-inches to 12-inches.

This paper discusses the planning and design of the intake facility, which is a 180-foot deep, 16- to 20-foot-diameter vertical shaft connected to the lake via a single 48- to 72-inch diameter microtunneled intake tunnel with a lake tap. A surface-mounted sloping intake with seven ports will allow water to be drawn from various depths of the reservoir for optimal water quality control.

Also addressed are the intake alternatives considered during conceptual and preliminary design, and detailed construction planning with the use of a geotechnical baseline report (GBR) in the construction contract documents.

Introduction

Located in the central coast of California, the San Luis Obispo County Flood Control and Water Conservation District (District) is implementing the Nacimiento Water Project (NWP), a raw water conveyance system to deliver 15,750 acre-feet annually from Lake Nacimiento to participating agencies including City of Paso Robles, City of San Luis Obispo, Atascadero Mutual Water Company, Templeton Community Services District and County Service Area 10, Zone A (Figure 1).

In 2005, Black & Veatch Corporation, Irvine, California, was selected to perform preliminary and final design of the $185 million project, which is scheduled to be in operation by 2010. A key element of the Project is the NWP Intake, consisting of a 20-foot diameter, 180-foot deep, concrete-lined vertical shaft connected to the Lake via a single 530-foot long, 48-inch diameter micro tunnel with a lake tap. A surface-mounted sloping intake with seven ports will allow water to be drawn from various reservoir depths for optimal water quality. Construction is scheduled to begin winter of 2007.

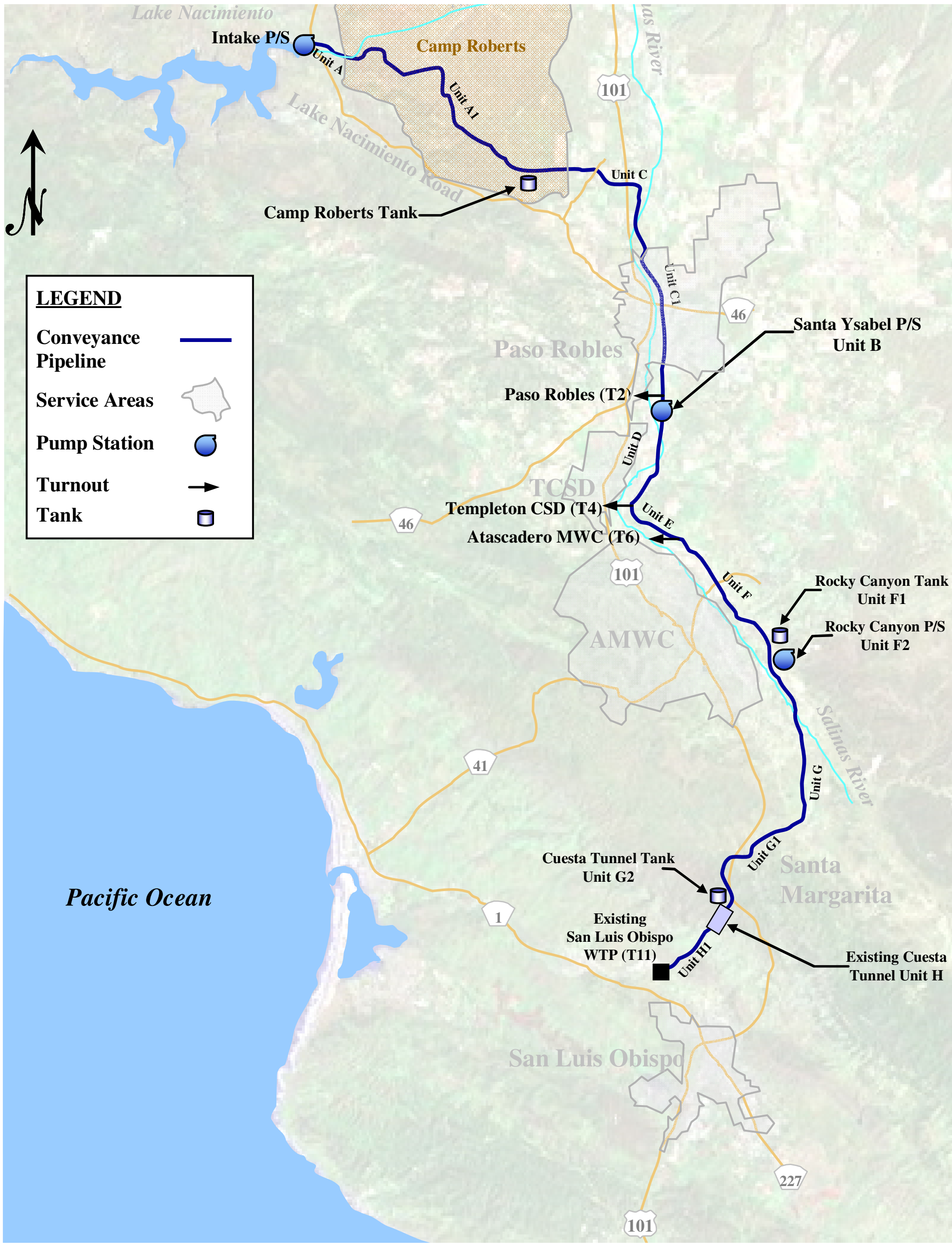
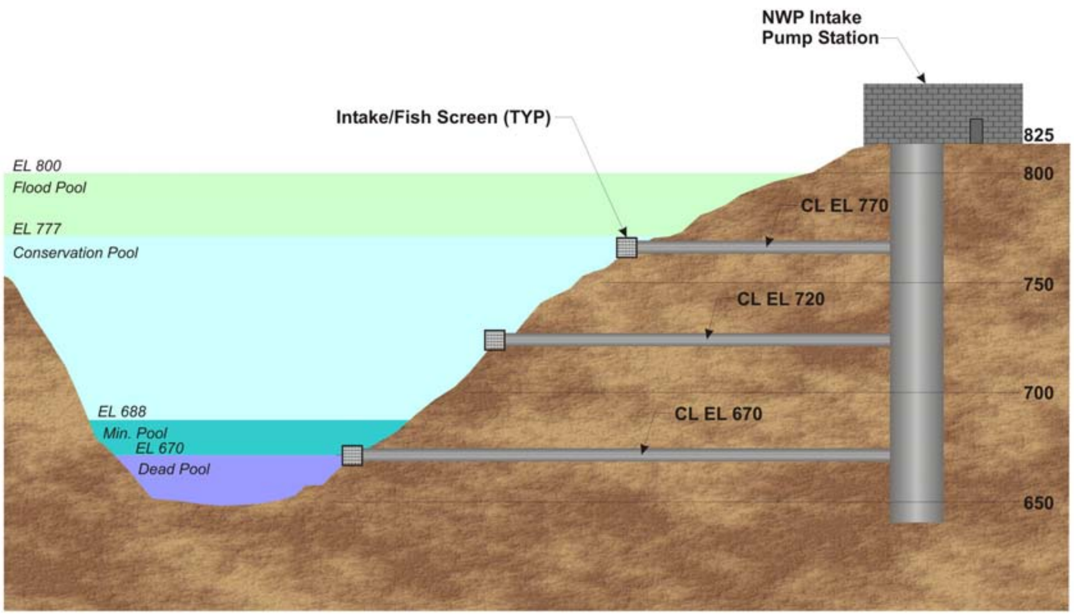


Figure 1. Nacimiento Water Project, San Luis Obispo County, California

NWP Intake Alternatives

The Nacimiento Water Project Final Environmental Impact Report (FEIR), December 2003, represented the conceptual design of the intake facility as a fixed three-port lake intake, with each port connected to the lake via a 72-inch diameter inlet tunnel (Figure 2). As design development progressed, it became clear that the fixed-port intake did not offer enough flexibility to withdraw raw water from varying lake levels (Figure 3) sufficient to optimize the water quality in combination with the participating agencies' treatment processes. A water quality investigation and review of intake alternatives was subsequently conducted by Black & Veatch in Dec. 2005.



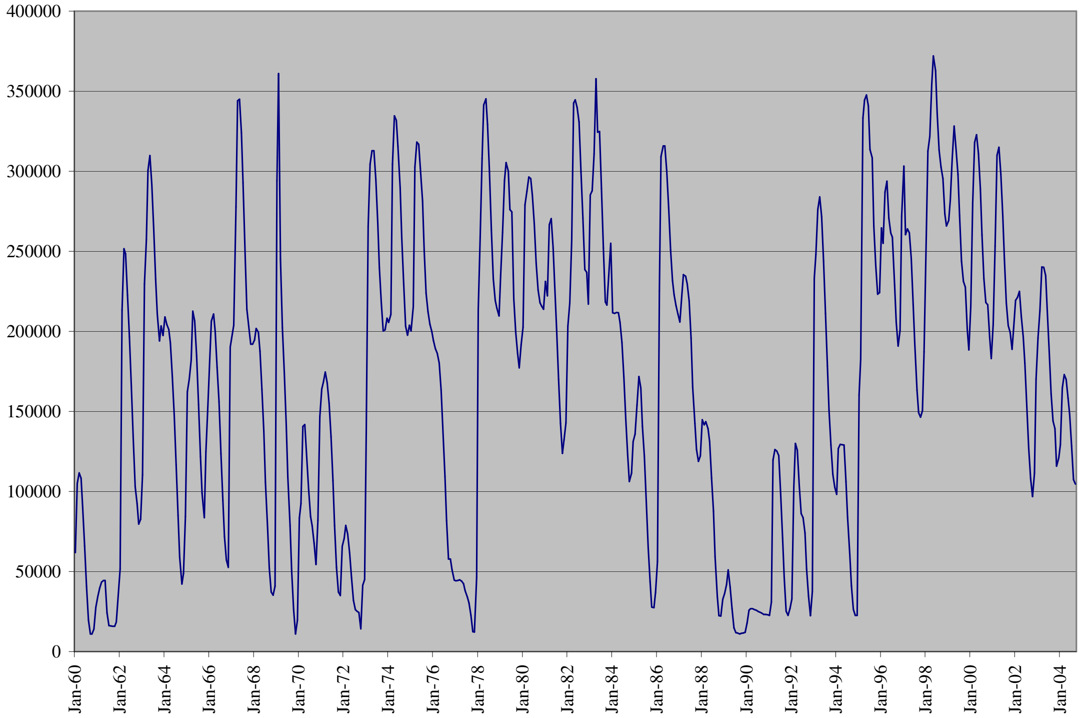
Source: Black & Veatch Corporation, December 2005

Figure 2. NWP Intake Configuration in the EIR

A technical memorandum (TM) was prepared to summarize the existing lake water quality data and to provide recommendations on intake port depths, chemical feed options for the raw water supply, and water quality monitoring. Black & Veatch reviewed water quality data for Nacimiento Reservoir obtained from the District, as well as historical water levels, for the purpose of analyzing the following lake characteristics:

- Determining the position of the thermocline in the water column;
- Describing water quality in the epilimnion and hypolimnion and identifying locations (depths) where water quality changes occur; and
- Identifying additional water quality data and monitoring needs.

Figure 3, obtained from the California Department of Water Resources website, shows historical reservoir storage volumes based on acre-feet of storage, which was converted to reservoir elevations for the analysis.



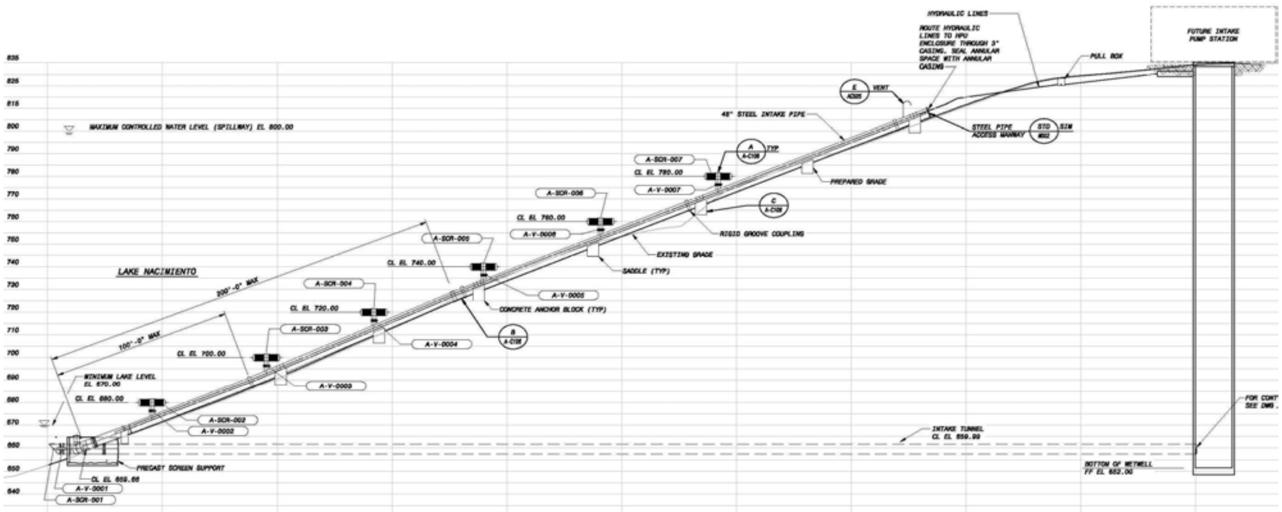
Source: California Department of Water Resources

Figure 3. Nacimiento Storage Levels, Acre-Feet (1960 to 2004)

As expected, the water quality data indicated that during the winter months the temperature profiles in the reservoir are fairly uniform and stratification is not present. Beginning in February or March, the surface waters start to warm and a thermocline starts to form. Usually during May, epilimnion, thermocline, and hypolimnion areas become distinct. The surface waters reach their highest temperatures in July and August and then start to cool. As the surface water cools, the thermocline erodes and the temperature profiles again become uniform. Stratification disappears as early as October or as late as December.

During the periods of a well established thermocline (May through September), the top of the thermocline was observed to be at depths of 15 to 30 feet and the bottom of the thermocline was at depths of 30 to 55 feet. The thermocline was usually 15 to 25 feet thick.

Based on the water quality data results, the number and spacing of intake ports were also reviewed as part of this TM. Ultimately, the decision was made to incorporate seven (7) intake ports spaced at approximately twenty (20) feet vertically in order to provide the flexibility to withdraw optimal water quality for any operating lake level. Figure 4 shows a schematic of the intake configuration accepted for final design.



Source: Black & Veatch Corporation, December 2006

Figure 4. NWP Intake Configuration Accepted for Final Design

NWP Intake Design

Final design of the NWP Intake focused on construction of the four principal features of the intake, namely, vertical shaft, intake tunnel, sloping intake and intake ports, and marine works. The following discusses the considerations involved with each part.

<u>Geologic & Geotechnical Considerations</u>

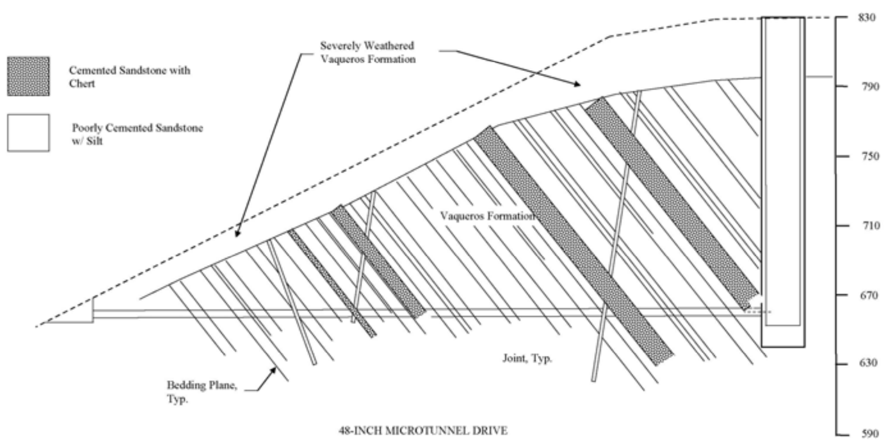
The intake site is located in the central California Coast Ranges Geomorphic Province, characterized by moderately rugged terrain and north-northwest trending ridges and intervening alluvial valleys. The site is located in a tectonically and seismically active region dominated by the San Andreas Fault System.

As outlined in the Geotechnical Baseline Report (GBR) for the intake, the intake shaft is located upstream of the existing Nacimiento Dam and adjacent to the north abutment and spillway. Geologic mapping indicates shaft and tunnel construction will take place in the Vaqueros Formation consisting of moderately lithified, massive, poorly- to well-graded sandstone. The formation is predominantly quartz with minor constituents that include feldspar and clay minerals. The formation can include conglomerates and granitic boulders, as well as thin partings of clay, claystone, or siltstone that separate the massive sandstone beds.

The Vaqueros Formation observed at the boring locations varies significantly in strength, hardness, and quality. In general, the formation consists of fine- to medium-grained sandstone whose mineral constituents are dominated by hard, abrasive minerals such as quartz and feldspar. Significant clay minerals ranging from clay-size to silt- and fine-sand size particles are also present in the formation.

One of the most significant characteristics of the Vaqueros Formation as relates to shaft and tunnel construction is the presence of hard, abrasive minerals. Published literature indicates quartz content of 50 percent to more than 90 percent. Grain size analyses on samples of the sandstone indicate the presence of significant clay-, silt- and fine-sand size particles, ranging from between 10 and 25% percent passing the No. 200 sieve. Much of this fraction of "fine" material, however, includes quartz and feldspar.

The structure of the Vaqueros Formation includes both bedding planes and two orthogonal sets of joints. As shown in Figure 5, the bedding dips steeply into the slope (north-northeast) at angles of 50 to 78 degrees. The nature of the bedding planes is highly variable which typifies sedimentary deposits. In general, the bedding planes are characterized by very thin (< 1mm) partings containing silt and clay. Elsewhere, the silt and clay can be absent entirely.



Source: Black & Veatch Corporation, Intake GBR, December 2006

Figure 5. Preliminary Geologic Section through NWP Intake

Joints are typically widely spaced, tight, and contain either precipitates of calcium carbonate ("healed") or evidence of weathering. Joints dip steeply from near vertical to between 45 and 80 degrees.

The intact strength of the rock varies between extremely weak and weak to moderately strong. Measured rock strength typically ranges from several hundred to several thousand pounds per square inch (psi), although lower and higher strengths were measured. The lowest measured uniaxial compressive strength was about 10 psi, while the largest measured values were on the order of 7,000 psi.

Ground water levels at the site will vary substantially with seasonal variations in precipitation within the watershed and with reservoir levels.

Shaft Design

Excavation for the intake shaft will extend approximately 180 ft ± below ground surface and will require excavation through a combination of fill, residual soil and weathered rock, and weak to moderately strong, intact rock. Plans for the project require a finished interior shaft diameter of 16- to 20- feet, with an excavated shaft diameter to be determined by the general contractor based on structural requirements shown in the contract documents and means and methods used for initial support.

Design of initial support for the shaft excavation will be the responsibility of the contractor. Initial support may consist of liner plate and steel ribs, steel ribs with timber lagging, slurry panel walls, secant piles, or casing installed using drilling methods, such as blind auger drilling. Since slurry panel walls, secant piles, and blind auger drilling provide pre-support of the ground prior to substantial excavation, these systems offer significant advantages over support systems requiring excavation prior to installation, including improved management of ground water inflows and significant reductions in excavation volumes.

Due to the unfavorable orientation of bedding planes within the Vaqueros Formation and the generally weak nature of the rock, the material is expected to squeeze and fast ravel within a cycle of bench excavation and erection of initial support. Thus, excavation volumes 35% greater than those corresponding to a “neat” line would be anticipated for support systems requiring excavation prior to installation. In addition, for liner plate and/or steel rib installations, ground water inflows and corresponding requirements for treatment and disposal must be considered. Based on our analyses, sustained ground water inflows of 500 gpm should be anticipated. Flush flows of up to 2,000 gpm are possible and if encountered are to be grouted to reduce total flows into the shaft to a 500 gpm threshold.

Microtunneling

A single 530-foot long microtunneled intake tunnel will connect the shaft to the lake with a lake tap. The construction method will involve jacking a steel pipe casing following a microtunnel boring machine (MTBM), with the casing serving as initial support and final liner. Plans for the project require a finished intake tunnel diameter of 48- to 72-inches, with the actual diameter to be determined by the general contractor based on MTBMs available at bid time and allowable jacking space resulting from the selected shaft diameter. The contractor will be responsible for selection of the appropriate MTBM and casing thickness to carry the thrust of jacking forces and other loads.

The MTBM will be driven from the shaft to the reservoir and retrieved “in the wet”. The retrieval of the MTBM will be staged from an excavation into the slope of the reservoir side wall that will be prepared prior to initiation of microtunneling.

The MTBM will be a closed, pressurized face, steerable, laser-guided, articulated tunnel shield capable of exerting continuous, controlled pressure at the tunnel face to prevent uncontrollable groundwater inflows and ground movements into the cutter chamber, with a reversible cutterhead drive system to minimize rotation of pipe during installation. It will also be capable of handling the various anticipated ground conditions to minimize loss of ground during tunneling and steerable and capable of controlling the advance of the heading to maintain line and grade within the specified tolerances. It will include a system to inject lubricant over and around the rear of the MTBM to reduce jacking friction and a slurry system to balance ground and groundwater pressure up to 140 feet of hydrostatic head.

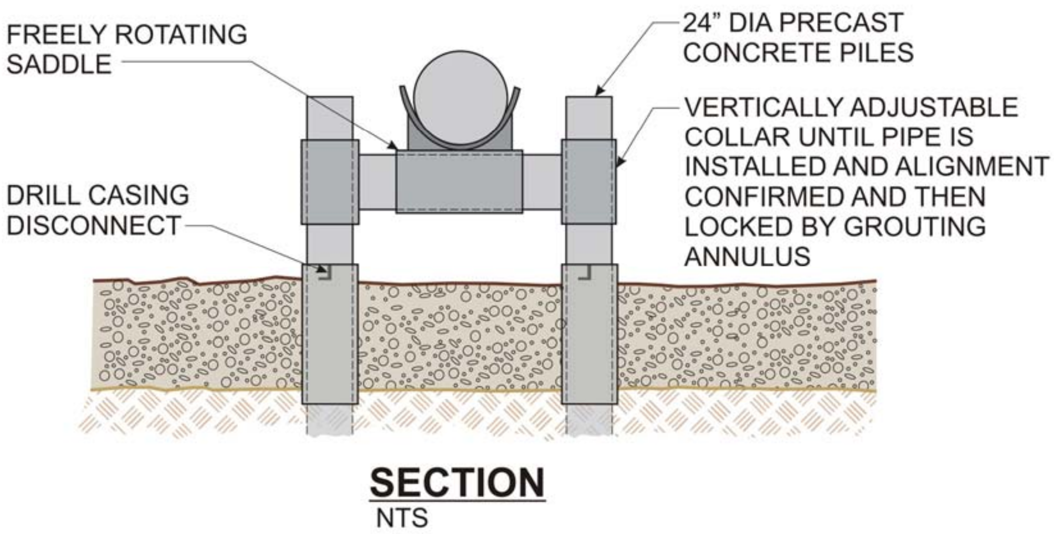
The overall tunneling system will also include a casing jacking system; launch seal affixed to the shaft wall and through which the MTBM and steel pipe passes; equipment to maintain proper air quality in case the contractor selects manned microtunnel operations during construction; lighting fixtures in watertight enclosures; and possible air lock to assist in cutter changes where changing cutters under atmospheric pressure is infeasible. The steel casing pipe will be either all welded steel pipe or Permalok pipe with gasketed joints.

The final push of the MTBM into the reservoir, the “lake tap”, is expected to be the riskiest part of the job. During lake tap operations, the contractor’s principal focus will be on safety of the work and personnel. The contractor will select its means and methods for performing the lake tap, including the type of removable bulkheads and/or flood valves to be used to ensure that the work is protected from flooding and unexpected water inflows given the relatively high head working conditions. The sequence for temporary support and removal of the MTBM to the lake surface will also be a critical activity.

Sloping Intake and Intake Ports

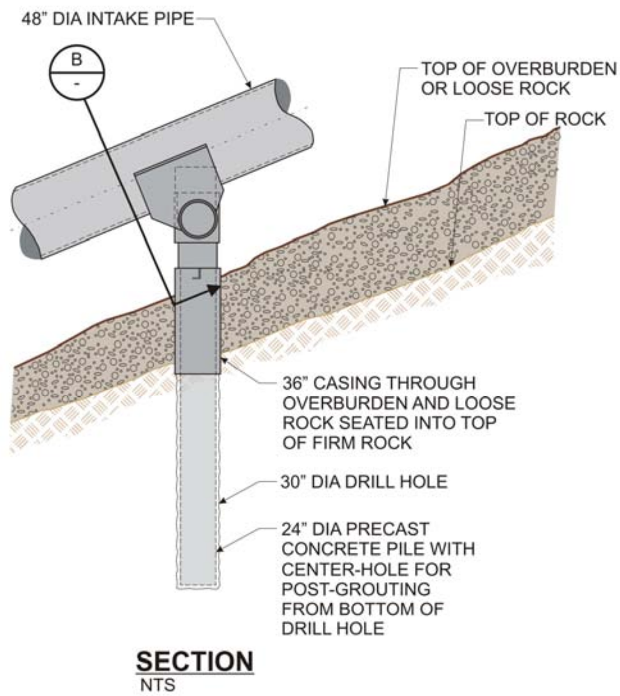
The seven-port sloping intake was selected to maximize the District’s ability to withdraw water from the best locations depending on actual reservoir water surface elevation, and time of year. The inlet ports are uniformly spaced vertically at 20-foot centers. The intake consists of a 400-foot long, 48-inch diameter, free-standing pipeline anchored on pipe supports; 24-inch diameter inlet ports with isolation butterfly valve and screen; and hydraulic system for intake valve operation.

A key construction planning activity focused on how to quickly and efficiently install the pipe supports and pipeline segments underwater from the lake surface. As shown in Figures 6 and 7, the intake design team, including underwater specialists from Ben C. Gerwick, Inc., devised a pipe support system that involves construction of cased drilled holes, followed by insertion of precast concrete piers and subsequent grouting to solidly lock the piers in-place. The individual pipeline segments (50 foot each) will be connected into 100- to 150-foot lengths and lowered down onto the pipe supports and connected to the pier tops with a fabricated steel pipe saddle. The saddle details will be finalized prior to advertisement for bidding, and the means and methods and final installation sequencing will be left to the installing contractor.



Source: Ben C. Gerwick, Inc., December 2006

Figure 6. Preliminary Section through Intake Pipeline Saddle Support



Source: Ben C. Gerwick, Inc., December 2006

Figure 7. Pipe Support Foundations for NWP Intake Pipeline

Marine Construction

Marine construction activities will support the installation of the sloping intake and retrieval of the MTBM. Associated with the marine construction will be the replacement / relocation of the existing log boom in the intake / spillway area.

Marine construction will involve establishing a floating marine operation with barge/crane and access to shore; diving operations; fuel transfer; underwater excavation; placement of tremie concrete; underwater construction of pipe supports; and underwater placement of pipeline segments, valves and screens.

Summary / Lessons Learned

Although construction cost was a key factor, the NWP Intake design evolution was eventually driven by water quality requirements – to provide an intake with seven ports that will allow water to be drawn from various depths of the reservoir for optimal water quality control. As a result, a surface-mounted sloping intake was adopted.

With this change in concept, underwater construction and placement of the intake pipe became a key focus of the design team. Details of construction sequencing and the design of pipe supports that are adjustable underwater were developed to assure the project is constructed in a safe and timely manner.

Advertisement to bid of the NWP Intake will occur in Spring 2007, and construction is scheduled to commence in late 2007.

References

Black & Veatch Corporation, *Nacimiento Water Project Technical Memorandum (TM) 8, Water Quality Investigations,* January 2006 (Final).

Black & Veatch Corporation, *Nacimiento Water Project, Intake - Geotechnical Baseline Report,* December 2006 (Draft).

Marine Research Specialists, *Nacimiento Water Project, Environmental Impact Report (Final)*, December 2003, prepared for the San Luis Obispo County Department of Planning and Building

Pipe Selection Criteria for Trenchless Projects: Microtunneling to Large Tunnels

Hugh Blockside [1] and Jozef Zurawski [2]

In the evaluation, planning, and design of pipes to serve as final liners for tunnels, the greatest emphasis during the design phase of the project is typically placed on the geological conditions to be encountered in the construction of shafts and the installation of the tunnel. The design of the shaft supports, selection of tunneling methods, initial tunnel supports, and control of the groundwater during tunneling become the most investigated elements in the design process. Often overlooked in the literature or in design guidelines are detailed descriptions of the processes necessary for proper pipe selection and pipe strength. If the wrong type of pipe is selected, damage during construction is possible and the system may have a less than planned service life.
Using five projects as examples, this paper attempts to identify key criteria in the selection of pipe for tunnels constructed by tunnel boring machine (TBM) and microtunnel boring machine (MTBM) methods.

In selecting the pipe for a given project, the following parameters must be considered:

- Service fluid type (i.e. potable water, sanitary, or storm sewer)
- Internal operating fluid pressure
- External ground and live loads
- External water load (groundwater)
- Method of installation
- Material handling loads
- Owner history with pipe material
- Construction costs
- Lifetime Expectancy.

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Service Fluid Type

Typical service fluids that flow through tunnels include raw water, potable water, wastewater, stormwater, and treated wastewater effluent from a treatment facility. Regulatory requirements can limit the choice of a pipe material because not all pipe materials are approved for certain fluids and for certain situations. For example, wastewater can become septic, depending on the source and the flow conditions (i.e. low velocity during dry periods), which in turn leads to the generation of hydrogen sulfide and a corrosive atmosphere in the pipe. Fiberglass (Centrifugally Cast Fiberglass Reinforced Polymer Mortar Pipe - CCFRPMP), polymer concrete, PVC, and HDPE pipe are generally required for a corrosive atmosphere. Also, not all pipe materials available for use in tunnels are acceptable for potable water use.

Internal Operating Fluid Pressure

The internal pressure of the fluid affects not only the choice of the pipe material, but the type of joint that can be used. In a tunnel where wastewater is flowing by gravity, the standard joint for a gravity sewer pipe may be sufficient. However, in some cases, it is the external ground (i.e., dead load), live load, and groundwater pressure that will dictate the pipe design and selection. Gravity sewer pipes of the following materials with standard joints are generally adequate.

- Reinforced Concrete Pipe (RCP)
- Vitrified Clay Pipe (VCP)
- Polyvinyl Chloride (PVC) Pipe
- Fiberglass Pipe (CCFRPMP)
- Polymer concrete pipe

In tunnels where gravity flow is anticipated, internal surcharging may often occur. Proper hydraulic analysis investigating a surge occurrence and its behavior in the tunnel (mostly performed by computer modeling) will estimate the internal surge pressure that can be expected within the pipe. This surge pressure needs to be checked against the pipe joint capacity that is usually established through testing performed by the pipe supplier/manufacturer. Such testing may be required specifically for some pipe joints to determine their capacity if such data is not available. This is necessary to establish if the proposed pipe will meet the internal operating and surge fluid pressures anticipated on the project. In general, if the fluid pressure exceeds the design limits of a standard gravity sewer pipe joint, then a pipe that has pressure rated joints is required for the project. Generally, this type of pipe can be:

- Concrete pressure pipe
- Steel pipe (bolted, welded or Permalok© type joints)
- PVC pressure pipe
- Fiberglass (CCFRPMP) pressure pipe
- Ductile Iron Pipe

Sometimes CSO tunnels are designed assuming gravity flow for a given storm event. Sometimes this is not coordinated with nature and a less frequent storm occurs putting the system shafts and tunnels under pressure. A gravity pipe should not be used in these cases.

A watertight pipe joint for the tunnel is of utmost importance for tunnels constructed in non-cohesive soils. A leak in a joint may lead to a loss of soil surrounding the tunnel which in turn will lead to surface settlements, sink holes, pavement collapses, tunnel collapse, or other subsurface and surface utilities damage. This loss of fine soil particles around the tunnel may occur either by soil infiltrating into the tunnel or by fluid pressure piping through the ground carrying the soil away from the area around the tunnel (pipe).

In water tunnels, high water hammers can be expected at the transition of vertical and horizontal pipes. The selected pipe material should be designed with higher strengths than the rest of the tunnel.

External Ground and Live Loads

In most cases, the external ground and live load (as well as the groundwater or hydrostatic pressure) determines the strength requirements for the tunnel pipe (or final lining of the tunnel). Most tunnel pipe types can be designed (generally by wall thickness or by additional reinforcement) to support the loads imposed on the tunnel. The resulting stress and the joint type required will eliminate the selection of some pipe types and materials. The design of the pipe for the external loading will be the deciding factor in most cases. The ground conditions and live loads imposed from a railroad or highway will generally determine the controlling stresses. In shallow installations, especially for large diameter pipes, even the ground load alone in poor soil conditions may prove to be the controlling factor.

External Water Load

The external water load (groundwater or hydrostatic pressure) contributes to the determination of the type and strength of the pipe material required. It also dictates the pipe joint type for deep installations where infiltration must be minimized or as in cases of treated potable water tunnels, totally eliminated. For steel water pipes in deep installations, the external groundwater pressure is often the deciding factor in determining the thickness of the pipe wall because pipe failure by buckling can occur when the tunnel is emptied to conduct routine inspections.

Method of Installation

The method or means of installing the tunnel also contributes to the selection of the tunnel pipe material. Pipe may be pushed (jacked) into place (i.e. pipe jacking or microtunneling), or it may be placed on a pipe carrier and carted into place for the final placement, as in conventional tunnel construction. If a pipe is pushed into place, the pipe joints and the pipe wall must be capable of handling the longitudinal loads placed on them. Not all pipe and joints are suitable for jacking. The weight of the pipe and the joint bearing capacity will determine the maximum length that a pipe can be jacked. The economics of the placement may favor one type of material over another. The use of an intermediate jacking station may be required to keep the stresses within the pipe capacity limits. Typical types of pipe installed by jacking or microtunneling are RCP with special joints, polymer concrete, vitrified clay microtunnel pipe, steel, concrete pressure pipe and CCFRPMP products.

Material Handling

The length of the pipe segment, its material, the weight of the pipe, and the pipe diameter will be the factors used in analyzing stresses resulting from pipe handling during transportation, lifting and lowering it, and moving it within the tunnel. The material handling stresses together with limitations by highway and bridge loads may limit the weight and, in turn, the length of an individual segment of pipe to be used on the project. In tunnel construction, the length of the pipe segments has a direct bearing on the construction cost because the number of pipe joints has a direct impact on the pipe material, pipe handling and placement costs (i.e. if only one pipe segment can be transported on a truck, the final price per foot of pipe is increased). Cost per foot of pipe associated with pipe handling at the shaft and transportation through the tunnel increase when larger numbers of pipe segments are needed to complete the tunnel. This is also true for jacking or microtunneling installations where more joints will also result in higher cost of the project. The length of the pipe sections selected for the tunnel project will also impact shaft sizes and their costs. In restricted urban areas, shafts may have to be smaller than desired, which will necessitate smaller pipe segments.

Owner History with Pipe Material (Design and Selection Requirements)

In many cases, more than one type of pipe will meet the design criteria to serve as the final tunnel liner. In some projects, the owner's history with a given pipe material may become a major factor that can influence the selection. The owner's good experience record with a certain type of pipe's performance and service may be an indication of a quality product available within the local pipe market. Unless technically not feasible, such a pipe type should be seriously considered for the project. However, issues such as installation of the particular pipe in the restricted space of the tunnel environment, while allowing for proper joint mating, must be carefully evaluated. This review must include a study of case histories of installations of this specific pipe manufacturer in underground conditions. Some pipe joints are easily mated in open cut installations, but pushing the joint "home" may require special arrangements in the tunnel. Some joints require a large force to mate the pipe sections properly. Most damage to pipes occur during transportation and installation. Pushing the joint "home" on a sharp curve or correcting a tunnel misalignment by deflecting the joint often results in damage to the pipe and loss of a pipe section or two.

In general, the most critical factors in selecting the pipe for use in the tunnel are:

- Fluid service – corrosive or non-corrosive and potable and non-potable
- Fluid pressure – type of joint (limits on infiltration and exfiltration)
- External loads – live loads, dead load, and groundwater pressure
- Method of Installation – pipe carrier or pipe jacking.

These four factors generally control the type of pipe used on a given project. Proper design of the pipe will assure a long lasting product that should perform properly for 50 to 100 years. However, too often there is a lack of proper independent design, and engineering judgment by the designers. More often than not, the pipe is selected based on the manufacturers catalog data without any additional analysis, testing, or design calculations. The pipe is seldom checked independently by the engineer for all possible handling and service conditions, or even final service loads. In fact, some of these tasks are often left to the contractor or supplier.

Construction Costs

In most tunnel construction with pipe as the final tunnel lining, the pipe material cost varies from 5 to 20 percent of the total tunnel construction costs (per linear foot). In large diameter tunnels, the pipe material cost is the smaller fraction of the total cost. In some short microtunnel installations, under difficult ground conditions, the pipe cost may become somewhat incidental to the cost of the entire installation. Based on this, it can be seen that a poor selection of the pipe for a specific project will result in problems during installation that far outweigh the cost of the pipe. For this reason, pipe selection should always be based on sound technical requirements and proper design that includes all construction and handling loads.

If properly designed from a structural standpoint (all loads identified and included in the design) the pipe will not be overstressed during installation. A successful installation will guarantee a long lasting product with tight joints and trouble free service for many years. This is of utmost importance in soft ground tunnels, where the loss of soil from around the pipe joint may result in uneven loadings or loss of support around the pipe. This may result in structural failure, ground and surface collapse, and/or damage to other utilities. In such cases, the repair will require costly ground improvements and structural rehabilitation with interruption of service.

In pipes installed by jacking (pipe jacking or microtunneling), problems that arise are usually associated with pipe joints. Overstress on the joint during jacking and during grade or alignment corrections will damage the joint, which will eventually lead to leaks, soil infiltration, and failure. Again, proper pipe selection during the design and inspection of the installations with qualified personnel following strict guidelines for pipe acceptance or rejection during manufacturing, delivery, and construction must be part of every project. Repair of a joint in the microtunnel installations in soft ground is difficult and almost impossible and joint repair sleeves must be used on the inside of the pipe for most of the repairs.

Lifetime Expectancy

Once the installation is successful, the biggest factor in the life of a pipe is the wear and tear on the interior pipe wall by the service fluid. Corrosion and erosion can deteriorate the internal surface of the pipe and, eventually, affect the structural stability of the pipe. Matching the pipe material to the service fluid can minimize or eliminate erosion and corrosion, thereby extending the life of the pipe.

As a result, the pipe selection for tunnel installations should never be based on the cost of the pipe material alone as one of the primary selection factors. The pipe must meet the service, structural strength, and installation requirements for the project first. Proper independent structural designs must be performed to select the pipe material type for the project so that it will perform under all anticipated hydrostatic, live, dead, and service loads and conditions.

CASE HISTORIES

Toledo, Ohio - Eastside Water Main Contract B

Contract B of the Eastside Water Main project was one of a number of projects designed to improve the reliability of Toledo's water distribution system and to extend the service area. Contract B consisted of extending the water line across the Maumee River from Lucas County to Wood County in an alignment that is adjacent and parallel to the Ohio Turnpike (I-80 & I-90) bridge across the river, and adjacent and parallel to, an unused railroad bridge. In order to meet the hydraulic requirements of the project, a 48" diameter water main was needed. The water main was designed for a working pressure of 115 psi in tunnel, with a test pressure of 150 psi.

The crossing of the Maumee River was made by tunneling approximately 1,600 LF through a non-karstic limestone, with a construction cost of $4.8 million. The geotechnical investigation revealed the presence of fractures in the limestone; therefore the possibility of encountering groundwater during the tunneling was a possibility. The excavation of the tunnel was specified to have a 94" minimum diameter and a 114" maximum diameter. The tunnel supports consisted of rock bolts with steel straps, with provisions for gasketed liner plates and grouting if wet conditions were encountered. The tunnel profile was selected to have 25 feet of rock cover under the river with a shaft on each side of the river. The shafts were 80 to 100 feet in depth through soft ground and rock.

Three types of pipe were considered for the project:

- Cement mortar lined steel pipe
- Pre-stressed concrete cylinder pipe (PCCP per AWWA C 301)
- Ductile Iron Pipe

Although all three pipes met the hydraulic standards of the project, the cost and owner's experience with the pipes were the driving factor in the selection of pipe type for this project. Steel pipe was estimated to be the highest cost pipe material for the project, but would require the smallest tunnel excavation diameter. Both ductile iron and concrete pressure pipes had been used within the city's distribution system. For this project, concrete pressure pipe was selected because of the City's past positive experience with this pipe type in the size range required for the project had been good and the pipe was readily available.

The tunnel itself was designed with a sufficient diameter to allow for installation of the PCCP with pipe-carrying equipment (jacking was not permitted to avoid joint damages). Once in place, the void between the tunnel wall and the pipe was filled with cement grout.

New York City, Croton Water Tunnels, Contract CRO-313

The CRO-313 project was designed to connect the new Croton Water Treatment facility with the existing New Croton Aqueduct tunnel. Twin 9' ID pipes in a single tunnel (one providing low-pressure service and one providing high-pressure service) were designed to convey treated water for distribution into the water system. The original design specified Reinforced Concrete Cylinder Pipe (RCCP) for the low pressure service line and cement mortar lined welded steel pipe for the high pressure service line. The low pressure service line was designed for a working pressure of 59 psi and a test pressure of 71 psi. The high pressure service line was designed for a working pressure of 111 psi and a test pressure of 200 psi. The pipes were specified to be installed one on top of the other in a single, large and tall horseshoe tunnel (Figure 1), to be excavated by the drill and blast method. The Contractor proposed alternative means and methods by excavating two parallel TBM tunnels (Figure 2) and presented these as a value engineering proposal with a credit to the Owner.

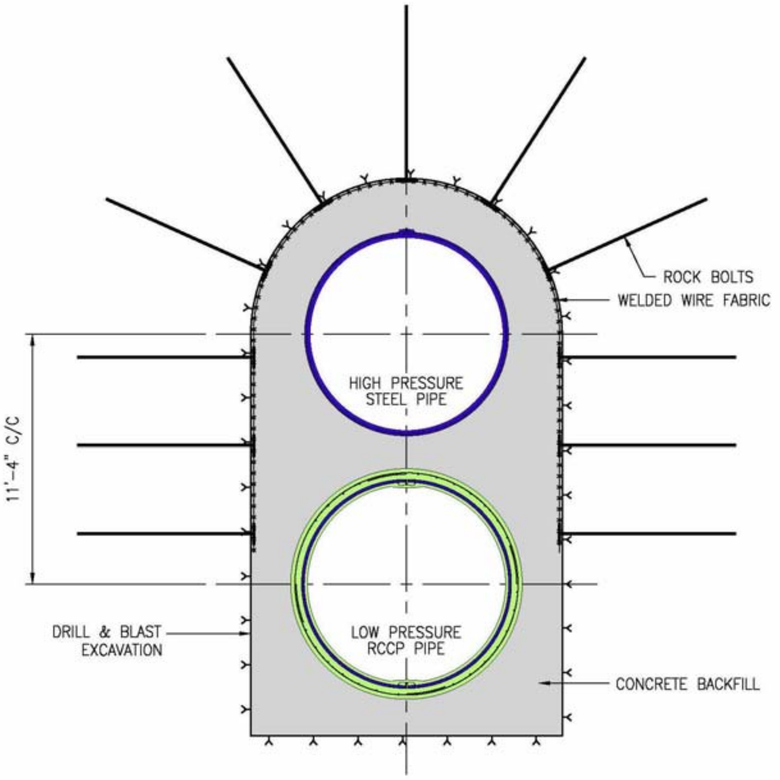

FIGURE 1

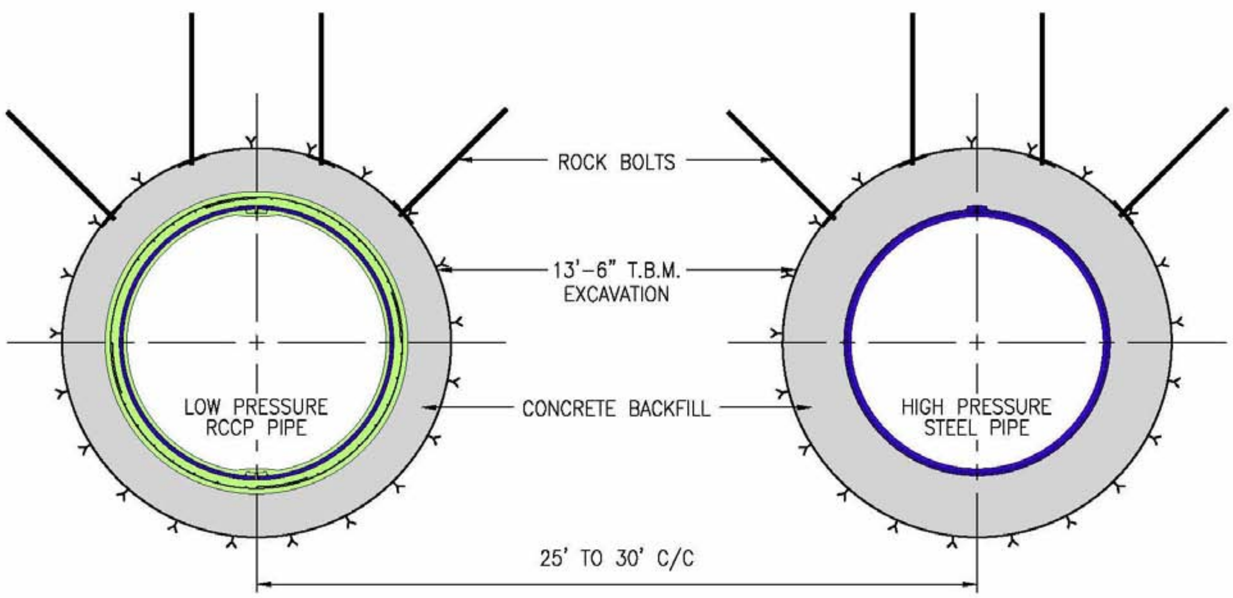

FIGURE 2

Each TBM tunnel would house a single pipe. Since changing the pipe installation arrangements from one on top of another into a single drive for each, the pipe interaction and ground loading on the pipes also was changed. Final ground loading on pipes installed in tunnels depends not only on the ground type but is also influenced by a method and the size of the tunnel excavation. Driving a TBM tunnel reduces the rock load on the pipe. TBM excavation results in a smaller opening and less disturbance to the surrounding rock. As part of the contractor's proposal, cement mortar lined steel pipe, as manufactured by Permalok using their "interlocking" (type T7) pipe joining system, was investigated for both the low-pressure and high pressure service lines.

The prime reason for evaluating the Permalok pipe and the type T7 interlocking joint in lieu of the welded steel pipe for the high pressure service line was the ease and speed of installation. In comparison, the specified full depth welded joint is very labor intensive, as each 12 foot long pipe section requires a 28 foot circumferential weld within the tunnel environment. With the Permalok interlocking joint system, the joint could be pushed "home" without significant delay while the delivery and setting of the pipe took place in the tunnel. For the low pressure service line, using the Permalok pipe instead of the RCCP resulted in a smaller tunnel excavation diameter. Another reason to use the Permalok pipe and the type T7 interlocking joint for either line was the ease of handling during installation as the steel pipe has a lower weight per linear foot than the RCCP.

The T7 joint is designed for high pressure service and has been successfully tested and used on projects with smaller diameter pipe (72 inch). However, the Owner of the Croton Project did not have any experience with the Permalok pipe and its joint for water service applications. Also, the supplier was unable to provide full scale testing of the joint in time without affecting the project schedule. As a result, it was the lack of experience and lack of proper engineering data for the pipe joint (in this pipe diameter and service pressure level) that ultimately lead to the continued use of the welded steel pipe for the high pressure service line and the use of RCCP for the low service line. A result of this decision to use the RCCP for the low pressure service line in the TBM tunnel, the diameter of the tunnel excavation was increased to 13'-6" to accommodate the installation of the pipe with a thicker wall.

As for the welded steel pipe for the high-pressure line, the wall thickness was selected based on the pipe buckling resistance against the groundwater pressure.

The change in construction method from a single drill and blast horseshoe tunnel to dual TBM tunnels was expected to provide a cost savings of approximately $8 million on this $100 million construction project.

Cleveland, Ohio Contract T-230 Northeast Ohio Regional Sewer District

The T-230 sewer project was designed as a wet weather relief sewer for a sanitary sewer service area that is subject to high rates of infiltration and inflow, resulting in basement flooding and sanitary sewer overflows. This approximately 11,000 LF long rock tunnel relief sewer was designed to discharge to the Southwest Interceptor (which is a tunnel). The major design concern from a hydraulic standpoint was the large variation between the dry weather and the wet weather flow rates. The dry weather flow rate in the proposed T-230 tunnel was low enough that the potential for septic conditions existed. This would lead to the generation of hydrogen sulfide, and ultimately sulfuric acid, which would ultimately result in corrosion of a concrete tunnel lining.

The project was conceived in the facilities planning process as a pipe in tunnel. During the design phase, it was determined that a 60" pipe at a slope of 0.40% was required for the wet weather flows. Since the sewer had a relatively small diameter for a tunnel project, a precast pipe was a logical choice for the final tunnel lining. The tunnel would be excavated by a TBM and the pipe would be installed after the excavation was completed. RCP with a cast-in-place low flow channel (cunnette), fiberglass (CCFRPMP), and PVC were the pipe materials evaluated for the project during the design phase. The PVC pipe met the hydraulic conditions of the project, but did not meet the strength requirements resulting from the deep rock tunnel (100 feet depth) installation. Both the RCP (Class IV Wall B) and fiberglass (62 psi stiffness, 1.27" wall thickness) met the strength requirement of the project. However, a cunnette in the invert of the RCP was required to eliminate the concerns about septicity, the generation of hydrogen sulfide and odor generation. The fiberglass pipe, of course, provided the required protection against the affects of hydrogen sulfide during low flow periods.

The contract documents offered the bidders two pipe choices: RCP with a cunnette invert or CCFRPMP. The successful bidder selected RCP with an invert cunnette already cast into the pipe sections at the manufacturing facility. Figure 3 shows the pipe with the cunnette section; Figure 4 shows the pipe installed in the tunnel. The bid cost was $7.5 million for the pipe in tunnel - $682/lf (shaft and surface construction costs not included).

FIGURE 3
RCP with Cunnette

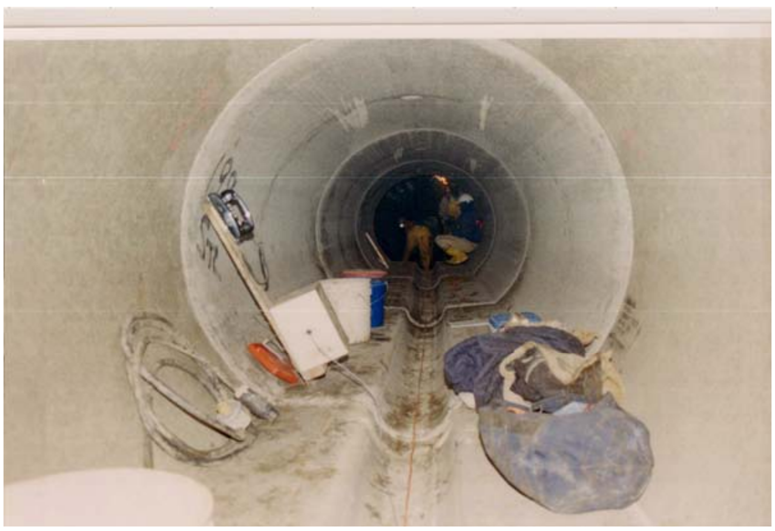

FIGURE 4
RCP with Cunnette installed in tunnel

Bagley Road Microtunnel - Olmsted Falls, Ohio

The Bagley Road Project consisted of an approximate 2,300 LF long storm sewer installed by the microtunneling method as part of a railroad (RR) grade separation project. The design pipe size ranged from the minimum acceptable diameter of 36" to the maximum acceptable diameter of 48". Any pipe size in this range provided the required flow capacity for drainage and adequate cover over the tunnel. The range of pipe sizes allowed the bidders to work with a wider choice of microtunneling machines available for the project. The microtunneling operation was to be performed in the Berea Sandstone, which is a very abrasive and hard rock with compressive strengths in the range of 10,000 to 20,000 psi. Microtunneling with pipe jacking was selected as the installation method because of the depth of the sewer and to avoid blasting for open cut trenches along the RR tracks. A key factor in the selection of the pipe was the requirement for a factor of safety of three (3.0) for the pipe to withstand the jacking thrust required for the MTBM to excavate the rock. Corrosion resistance, the ground loads, and leak free joints were not major factors during the design when selecting the pipe types for this installation.

Three types of pipe materials were evaluated during the design process; VCP, RCP, and polymer concrete. PVC and HDPE were the practical choices if the tunnel were to be excavated by conventional TBM methods, and if the pipes were to be installed in the tunnel after the excavation was completed. However, when evaluating the tunneling options, it became obvious that microtunneling appeared to be the most cost effective method for the geological setting. This conclusion resulted in specifying the type and strength of pipe that were suited for the microtunnel operation. The following minimum requirements for the pipes were specified in the Contract Documents:

- VCP (7,000 psi compressive strength with minimum three inch wall thickness per ASTM C 1208)
- RCP (Class V Wall B per ASTM C 76)
- Polymer Concrete (Class V per ASTM D 6783)

The above pipe types can allow for the microtunneling operations to be carried forward with minimal difficulty. The pipes were specified as jacking pipes, requiring the wall and the joint to carry certain minimum jacking loads identified in the contract documents.

The contractor for the project selected the Denlok VCP pipe for microtunneling, as manufactured by Can-Clay with a 36.3" ID and a 44.5" OD to match the selected MTBM.

Figure 5 shows the VCP jacking pipe and Figure 6 shows the VCP pipe being installed in the microtunnel.

FIGURE 5
VCP Jacking Pipe

FIGURE 6
VCP Jacking Pipe Being Installed by Microtunneling in Rock

Orchard Meadow CSO Relief Sewer, Youngstown, Ohio

The Orchard Meadow project was a 1,600 LF long relief sewer designed to eliminate a CSO discharging into Mill Creek within a park in Youngstown, Ohio. The narrow ROW along the creek and concerns regarding slope stability using open cut construction along a hillside resulted in the decision to construct the most downstream 800 LF segment of relief sewer in tunnel. Both RCP and fiberglass pipe (CCFRPMP) were considered for the project. Both were adequate from a hydraulic and a ground load basis. As this was a relief sewer for combined sewer flow (essentially an increase in capacity allowing the elimination of a regulator and overflow), the low flow characteristics of the pipe were a concern. Low flow (dry weather) rates were low enough that deposition of solids and the potential for hydrogen sulfide generation were concerns. As a result, fiberglass pipe was selected for use on the project. A 60-inch diameter pipe was selected as the carrier pipe inside a 114 inch tunnel, excavated by a TBM. The initial tunnel liner consisted of steel ribs and timber lagging or liner plates (depending on the ground conditions to be encountered). The steel ribs and liner plates were considered to be temporary tunnel support as the pipe was selected to carry all of the anticipated long term ground and water loads. After the tunnel was excavated, the pipe was placed in the tunnel and the annular space between the rock or initial temporary supports was filled with grout. As an option, microtunneling with pipe jacking was also specified. The project was constructed using a conventional TBM at a bid cost of $2.3 million. Figure 7 shows the fiberglass pipe being moved on the construction site. Figure 8 shows the pipe blocked in the tunnel.

FIGURE 7
Fiberglass Pipe being carried to the tunnel shaft

FIGURE 8
Fiberglass pipe blocked against the liner plates prior to backfilling with grout.

Conclusion

In designing a tunnel, the level of attention given to pipe selection should be the same as given to the tunnel design. The pipe design and selection effort must be complete and independent using state of the art design tools. The design must identify all anticipated critical construction and service loads that will be imposed on the pipe throughout its service life. The pipe material selected for the project must safely carry stress from all of the loading, as well as it must meet the joint and strength requirements. Proper selection of the pipe material and joints will result in a pipeline that will attain its desired design life expectancy.

The construction and service conditions of the pipe should be the primary factors in selecting the type of pipe and its joint requirements. Once these conditions are met, then price can be a consideration.

Utilizing Trenchless Technology for Design of Utilities in Baytown

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Abstract

Baytown, Texas is a highly industrialized city of oil refinery, rubber, and chemical industries located on Galveston Bay, approximately 30 miles east of Houston. Baytown population grew from about 20,958 in 1948 to about 66,430 in 2000, and its continued growth requires planning of city services including sewer service. In order to promote continued growth in the N. Main Street and Baker Road area, the City is planning to extend Baker Road, eastward from N. Main Street and provide sewer service to current and new businesses and neighborhoods in the area. The sewer project included 15-inch gravity sewers along N. Main Street, a duplex lift station, and a 6-inch force main crossing a flood control channel.

N. Main Street is a five-lane roadway, constructed of reinforced concrete. As the street runs through the heart of Baytown, it experiences a steady flow of heavy traffic. It would be difficult and costly to close portions of the roadway for the construction of the proposed gravity sanitary sewer. The numerous utilities within the Right-of-Way also posed challenges for the installation of a new sewer system. Within the N. Main Right-of-Way, there are existing pressure force main, gas lines, water lines, over head power lines, and various other utilities. During design phase, it was determined that the most efficient and cost effective way to address these issues was to install the sewer system by open cut where feasible, and use the combination of augering and horizontal directional drilling to avoid existing utilities and minimize impacts on above ground features.

This paper will present the challenges**,** planning and coordination efforts required to construction the sewer system describe the importance and benefits of trenchless technologies to Engineers during design and construction phases of infrastructure projects.

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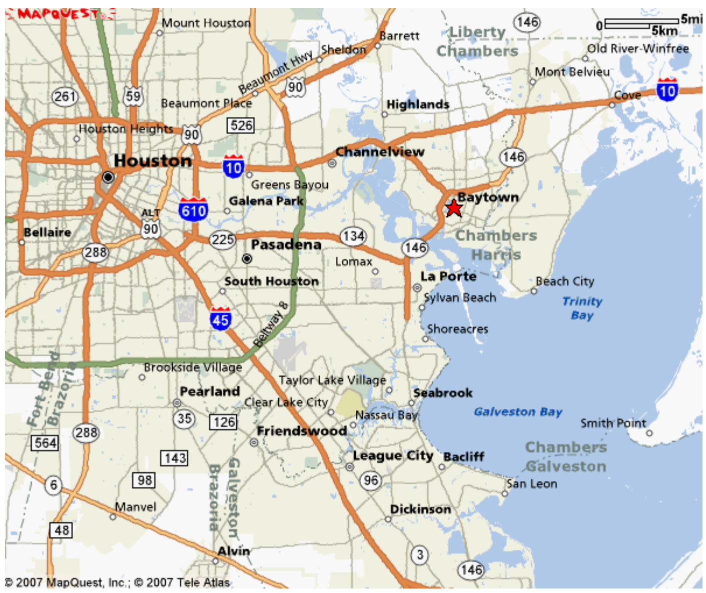
Baytown History

The first settlers in and around the current Baytown area appeared around 1822. Among them was Nathaniel Lynch, who set up a ferry crossing (which is still in operation today) at the junction of the San Jacinto River and Buffalo Bayou. This location has become known as Lynchburg. The settlers played an influential role in the battle for Texas Independence. It was during this time that the community of Goose Creek started to develop. In the late 1910's, a rival community was founded on the east side of Goose Creek, named Pelly. The community of East Baytown was created in the early 1920's. Over time, the `East' part of `East Baytown' was dropped, due to the fact that East Baytown was west of Goose Creek. After World War I, people began to discuss consolidating the three towns into one. The town of Baytown resisted this, however; finally, in 1947, the three towns consolidated. The town of Baytown as it is known today was founded on January 24, 1948.

In 1908, an oil strike was made beside Tabbs Bay. Oil exploration began in earnest, and in 1916**,** the Goose Creek oil field became the first offshore drilling operation in Texas (the second one in the nation). In 1919, the directors of Humble Oil & Refining Company built a refinery in Baytown. This refinery would become one of the largest in the world, and a very influential force not only in Baytown, but also during World War II. The refinery produced over a billion gallons of aviation fuel (100 octane gasoline) during WWII, and nearly half of the total butyl rubber production in the United States in 1945. Over the next 50 years, the population increased from 20,000 to almost 70,000.

Figure 1: Baytown Location Map



Baytown is located on the northern shore of Galveston Bay**,** between the San Jacinto River on the west and the Trinity River on the east. Please see Figure 1. This Coastal Plains section of Texas, which is within Harris County (of which Houston is the county seat) lies approximately 30 miles east of Downtown Houston 50 miles west of Beaumont, and 40 miles north of Galveston.

The ground elevation ranges between 0 and 30 feet above sea level. The tidal bays of Baytown, which spread to Galveston and Trinity Bays, produce an abundance of marine life.

Baytown is a prosperous industrial community with major corporations, such as ExxonMobil, Exxon Chemical, Chevron Phillips, Bayer, Amoco, and more. Retail sales have climbed each year**,** and have exceeded $600 million annually. Retail stores range from quaint little shops and boutiques to one of the largest single-level shopping malls in the Southwest.

General Project Description

Baytown has experienced enormous population growth in the past decade. This growth has placed a strain on city services such as sewer and water. The City of Baytown has responded to these needs by implementing a strategy to meet current and future demands through its Capital Improvement Plan (CIP). One goal of the CIP is to provide a previously under-developed area located along N. Main Street and Baker Road. This is a mixed use area with existing businesses, multi-family unit dwellings, residential subdivisions, and churches located along N. Main Street. Providing sewer and other services for this area will allow developers to construct new apartment complexes, residential neighborhoods, and commercial businesses along N. Main Street and Baker Road. To serve this need, the city is planning to begin a project to construct approximately 2,459 linear feet of 15-inch gravity sewers by open cut, 256 linear feet of 15-inch gravity sewers by auger, several new manholes, 215 linear feet of 6-inch force main crossing Harris County Flood Control District (HCFCD) channel by horizontal directional drilling, and a duplex lift station. The construction cost estimate is approximately $1.0 million. The design for this project was completed in February 2007, and construction is scheduled to begin in May 2007.

Gravity Sewers along N. Main Street

N. Main Street is a five-lane major thoroughfare with a 62-foot right-of-way (ROW), constructed of reinforced concrete. The street traverses in the north-south direction and through the heart of Baytown. It experiences a steady flow of heavy traffic throughout the day. There are existing 10-foot wide utility easements on both sides of the street. The utility easement on the east side of N. Main contains a multitude of utilities, including an 8-inch natural gas pipeline, fiber optic cables, and a 12-inch sanitary sewer force main. In addition, overhead power and telephone lines are present. The utility easement on the west side of N. Main includes a 10-inch water line, and a small diameter force main. In addition, several driveways cross the utility easement. Since all lanes on N. Main must remain open for continuous traffic, installing the 15-inch gravity sewer within the street ROW was not an option. After conducting multiple site visits and reviewing survey drawings, the city decided the proposed gravity sewers will be installed along the west side of N. Main by means of the open trench method for the majority of the length, and auger construction for the sewer sections under existing driveways and crossing under N. Main Street.

There are five (5) existing driveways, which are approximately 16-foot wide each, connecting to N. Main that will require gravity sewers to be installed by auguring. These existing driveways serve as the entrances and exits to an apartment complex, a church, a retirement medical center**,** and a roadway, which is the only access to an existing development. All the property owners along N. Main indicated the strong desire not to have their driveway damaged and disturbed by construction or to impede traffic flow through the properties. The entrance and exit 15'x17' auguring pits will be located on each side of the driveways to accommodate auger construction.

A similar layout was designed for the 95-foot portion of the gravity sewer line crossing under N. Main Street. The PVC gravity sewer line will be encased and protected within a 21-inch diameter steel casing pipe. Auguring will commence at the entrance pit on the west side of N. Main Street, and terminate at the exit pit on the east side. Photos of the augur crossings for N. Main Street and one of the existing driveways are shown in Figures 2 and 3, respectively.

Figure 2: N. Main Street Crossing

Figure 3: Driveway along N. Main Street

The Auguring method of pipe installation begins with excavating entrance and exit pits, and hydraulically pushing or jacking a casing pipe (in this case a steel one) through the ground. The spoils are then removed through the casing pipe using a continuous flight of augers. The process is conducted from an entrance pit and terminates at an exit pit. The auger is driven by a power source which transmits power to the cutting head of the auger. Sections of the auger are added, as the auger proceeds, until it reaches the exit pit. After the boring is completed, the auger is removed and the proposed gravity sewer is installed into the steel casing pipe. This method is limited to amenable soil conditions and relatively short distances. Typically pipe diameters range from 4-inch to 82-inch, with a driving length up to approximately 600 feet.

6-inch Force Main Crossing of HCFCD Channel

The proposed 15-inch gravity sewers along N. Main Street will convey flow to the proposed lift station located near to the HCFCD channel. The lift station will pump flow through a 6-inch force main and discharge into an existing manhole on the opposite side of the channel within an existing 16-foot wide utility easement. The existing easement runs east to west, and contains an existing 12-inch force main, which crosses the HCFCD channel as an aerial crossing.

Two options were evaluated for the force main to cross the 130-foot-wide, 12-foot deep flood control channel. The first design option considered was an aerial crossing, which would require two concrete support piers for the force main to cross the channel. For such a crossing HCFCD requires hydraulic modeling be completed as part of the design to demonstrate that the new structures in the channel would not affect the 100 year water surface level. In addition, a flow diversion plan would be required for the diversion of storm water around the construction site.

The second option was a design for horizontal directional drilling under the flood control channel, which requires specialized drilling equipment and sufficient above ground open area for placement of piping and heavy drilling equipment. After performing a construction costs comparison, it was determined that the directional drilling method for installing the force main was the more cost effective option. The 6-inch, 215 linear feet force main was designed to have a minimum 10-foot of cover from the bottom of the channel. High Density Polyethylene (HDPE) pipe, SDR 11, was selected for the force main due to its flexibility and durability. Figures 4 and 5 show the HCFCD channel with the existing 12-inch force main aerial crossing.

Figure 4: HCFCD with Existing 12-inch Force Main Aerial Crossing

Figure 5: HCFCD with Existing 12-inch Force Main Aerial Crossing

The horizontal directional drilling process begins with boring a small pilot hole underground with a continuous string of steel drill rod. The pilot hole establishes a path for the drill rod and subsequent location of the force main. The drill rod is rotated and pushed into the ground by hydraulic cylinders in the drill rig. After the rod is inserted in the ground, another rod is connected to form a string. The process is continued until the pilot hole has been bored to the other side of the channel. Electronic detection equipment from above ground is used to track the drill rod at approximately 25’ intervals, and transmit the rod location to the driller for path adjustment as necessary. A drilling rig, with sufficient hydraulic power, is used to complete the drill pipe and pull back operations. A rig capable of exerting 60,000 lbs. of pulling force to overcome frictional drag, capstan effect, and hydrokinetic drag is sufficient for this project.

Drilling mud, typically made of bentonite and additives, is used during the drilling operation to stabilize the path hole, cool the cutter head, reduce drilling torque, remove cuttings, and lubricate the pipe during pull back. When the bore head and rod emerged at the opposite side of the crossing, a special cutter, called a reamer, is attached and pulled back through the pilot hole.

The pullback is done with increasingly larger reamers to achieve the desired bore hole diameter large enough for the pipe to be pulled through. The bore hole diameter must be kept to a minimum, but it's typically 1½ times the outside diameter of the new pipe to avoid surface settlement. One of the key considerations in the design of the drill path is maximizing the curvature of the path, while remaining within the allowable work area. The degree of curvature is limited by the bending of radius of the drill rod and the pipe. After the last reaming operation, the pipe is attached to the reamer through a swivel and pulled through the hole. Since flexible HDPE is to be used for this project, the degree of curvature is limited only by the bending of the steel drill rod. HDPE was chosen for the force main because of its light, flexible, inert material offering superb corrosion resistance. The HDPE can be joined mechanically or heat fused to create a leak free piping system. The pipe is fused and placed on the exit side of the bore and the entire pipe length is pulled back continuously in one segment.

Agency Coordination

Coordination with HCFCD is required for the installation of the force main to go under the 130-foot-wide flood control channel located on the eastern end of the project area. Construction plans with standard HCFCD construction notes were submitted to HCFCD to ensure compliance with its regulations. Storm water hydraulic modeling and permits were not required by HCFCD since construction process and the proposed force main alignment will not impact the storm water flow within the channel. All construction work will be done outside of the flood control ROW. Not having to perform hydraulic modeling and acquire permits from HCFCD reduced the time and effort required during the design phase.

Conclusion

As technologies for trenchless products develop and advance, trenchless methods will become even more attractive for the future of infrastructure design and rehabilitation. This project used the auger and HDD trenchless construction methods, which gave the owner cost effective methods for providing sewer service, while minimizing impacts to surface features. Coordination between the Owner, Engineer, and Contractor during the design and construction phases is necessary for the success of the project.

Bibliography

Map Quest Baytown Map
http://www.mapquest.com/maps/map.adp?formtype=address&country=US&popflag=0&latitude=&longitude=&name=&phone=&level=&addtohistory=&cat=&address=&city=baytown&state=tx&zipcode=

Water Infrastructure for the 21st Century: U.S. EPA's Research Plans for Gravity Sewers

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Beginning in 2007, the U.S. Environmental Protection Agency's (EPA) Office of Research and Development (ORD) is planning to support a new research program to generate the science and engineering to improve and evaluate promising innovative technologies and techniques to reduce the cost and improve the effectiveness of operation, maintenance, and replacement of aging and failing drinking water and wastewater treatment and conveyance systems. This research program directly supports the Agency's Sustainable Water Infrastructure Initiative. This paper will outline the plans for addressing condition assessment and rehabilitation needs for gravity sewer pipes in the U.S.

In 2002, the EPA's Office of Water (OW) carried out a study to gain a better understanding of the challenges facing the Nation's drinking water and wastewater utilities. EPA published **The Clean Water and Drinking Water Infrastructure Gap Analysis (EPA-816-R-02-020)**, also known as the *"Gap Analysis*" report. The report identified several issues that raised concern as to the ability of utilities to keep up with their infrastructure needs in the future:

- Our wastewater and drinking water systems are aging, with some system components exceeding 100 years in age.

- The U.S. population is increasing and shifting geographically. This requires investment for new infrastructure in growth areas and "strands" existing infrastructure in areas of decreasing population.

- Current treatment may not be sufficient to address emerging issues and potentially stronger regulatory requirements.

- Investment in research and development has declined.

This paper addresses two major research areas identified as critical by EPA; condition assessment and system rehabilitation.

Condition assessment encompasses the collection of data and information through direct inspection, observation and investigation and in-direct 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. Research issues in this area relate to the collection of reliable data and information and the ability of utilities to make

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technically sound judgments as to the condition of their assets. Condition assessment also includes the practice of failure analysis which seeks to determine the causes of infrastructure failures in order to prevent future failures.

System rehabilitation is the application of infrastructure repair and replacement technologies in an effort to return functionality to a drinking water or wastewater system or sub-system. The decision-making process for determining the proper balance of repair and replacement is a function of the condition assessment, the life-cycle cost of the various rehabilitation options, and the related risk reductions.

Condition Assessment of Gravity Sewers

Since the passage of the Water Pollution Control Act Amendments, better known as the Clean Water Act (CWA), in 1972, the major focus of sewer system condition assessment and rehabilitation has been the reduction of infiltration and inflow (I&I). Requirements for sewer system assessments were codified in the Rules and Regulations for Sewer Evaluation and Rehabilitation (40CFR35.927) and stated that U.S. EPA construction grants could not be approved unless there was documentation that sewer systems contributing to municipal wastewater treatment plants were not exhibiting "excessive infiltration and inflow." (EPA, 1991)

Therefore, sewer system condition assessment and rehabilitation have focused primarily on the reduction of I&I. This has almost exclusively targeted condition assessment towards the reduction of excessive hydraulic loading in collection systems and at treatment facilities. (EPA, 1991) Most sewer system evaluations were driven solely to determine if excess flows were more cost effective to treat or remove. While I&I still plague our current wastewater collection systems, and is linked to our current challenges relating to sanitary sewer overflows (SSOs) and wastewater blending, the growing adoption of comprehensive asset management by utilities is broadening the focus of sewer system condition assessment. In addition to condition assessment that seeks to identify excessive hydraulic loading due to I&I, comprehensive asset management examines the likelihood that wastewater infrastructure will deteriorate and fail, and the consequences in terms of costs and the effect on the system's ability to deliver services to utility customers and meet a wide range of performance measures. (GAO, 2004)

As the focus of condition assessment continues to broaden to include targets beyond the reduction of excessive hydraulic loading due to I&I, sewer system inspection technologies and investigation approaches must evolve. More innovative technologies will take advantage of observation and detection technologies, such as sonar, laser, ultrasonic, and infrared, not traditionally applied to sewer system investigation. In addition, the deployment of these non-traditional technologies will be supported by emerging digital, modular, and robotics technologies to greatly expand the "reach" of sewer system inspection techniques.

Corrosion of wastewater collection infrastructure, especially concrete sewers, is a significant cause of deterioration and premature failure. When exposed to the internal atmosphere of gravity sewers which is characterized by high humidity and the presence of hydrogen sulfide, sulfuric acid corrosion negatively affects concrete surfaces, mortar, and metal reinforcement material. Given this universal challenge for wastewater utilities, this research program will look into innovative inspection technologies and condition assessment methods that address corrosion-related wastewater infrastructure issues.

Related to wastewater collection system inspection and condition assessment is the evaluation of sewer system security vulnerabilities. In today's climate, vulnerability assessment of the collection system should be an integral part of an overall system condition assessment program. A recent GAO report found that few utilities have or are planning to install monitoring or security devices to detect and prevent system intrusions. (GAO, 2006) Recent work, funded by EPA, has been conducted by the American Society of Civil Engineers (ASCE) and the Water Environment Federation (WEF) on monitoring systems and physical security enhancements, including security measures for wastewater collection systems.

State of the Technology

The ability to visually examine the internal condition of a gravity sewer using closed circuit television (CCTV) has been the most important development in the area of inspection and condition assessment, leading to the current operation, maintenance and rehabilitation techniques employed by wastewater utilities. Incremental improvements continue to be made to CCTV technology, and as electronic components become more affordable, this technology can be applied by most contractors and plumbers. Basic CCTV systems which can inspect small-diameter sewer and drain pipes can be purchased for less than $1500. Also, a standard investigation of a sewer line can cost a utility no more than $1 per foot. (WERF, 2004)

Most utilities have established fairly simple rating systems which use the results from CCTV investigations to make an overall assessment of each section of sewer being inspected. This rating can then be combined with other data and information, such as results from hydraulic evaluations, sewer location, known soil conditions, and operational records, to determine maintenance and system rehabilitation priorities. However, CCTV assessments are qualitative and rely heavily on the skill of the investigation personnel to make judgments on the condition of the sewer. Also, this technology does not provide quantitative data to determine variations in sewer dimensions, subtle deformations, or debris level. Also, CCTV does not permit assessment of pipe condition below the water line within a sewer. While many sewers can be inspected during dry weather conditions to minimize this limitation, most trunk sewers maintain a fairly high flow and diverting these flows for inspection purposes is difficult. (WERF, 2004)

The application of CCTV in combination with newer technologies is currently being used in sewer inspections. Sonar technology, which uses high-frequency sound waves, can identify defects, especially large cracks, in the wall of sewer pipes and because it is almost exclusively designed to work underwater, it can overcome one of the shortcomings of CCTV. Laser technology can be used to identify variations in sewer pipes above the water line. Comparison of laser images of the interior dimensions of a sewer over time can be an effective method to determine temporal deterioration. (WERF, 2004)

One potential research issue emerges from this analysis of the state of technology for condition assessment. Given the ubiquitous application of CCTV for sewer inspections and condition assessment, a state of the art evaluation and technology transfer product on optimized application of CCTV results for condition assessment should be generated. Wastewater utilities that are well known for their innovative application of CCTV should be identified and a best practices assessment and tool produced. In addition, this effort could include optimizing the use of existing and historical data and information, in combination with CCTV information to establish baseline assessments.

Research Questions

The following key research questions relating to gravity sewer inspection and condition assessment have emerged from the research planning conducted by EPA. These key research questions reflect critical gaps in our knowledge of the performance of innovative inspection technologies, our understanding of proven condition assessment techniques, and our ability to diagnose and predict infrastructure failures.

- Can emerging and innovative inspection technologies, for both sewer and non-sewer assets, be identified and demonstrated in field settings to improve our understanding of their cost-effectiveness, technical performance, and reliability?

- Can advances in remote monitoring and wireless technologies be applied to develop in-system and in-pipe sensor systems, including real-time data collection, reporting and assessment, to reduce confined-space entry requirements for sewer system inspection and investigation?

- Can correlations be established between the assessed condition and measures of the performance, operation, or internal environment of sewers and non-sewer assets which could lead to the use of innovative indicators to determine and track the condition of assets over time?

- Can standard technical guidelines, uniform data requirements and indicators be developed for condition assessment of sewers and non-sewer assets, including manholes, service laterals and pipe joints?

- Can technical guidance be developed for establishing an overall wastewater infrastructure inspection program, including inspection prioritization, inspection frequency, inspection type (physical vs. visual, maintenance vs. structural), inspection by asset type, and inspection cost-effectiveness?

- Can cost-effective and reliable methods for the identification and assessment of wastewater exfiltration be developed?

- Can the dynamics of wastewater collection system infrastructure failure be better understood, diagnosed, and learned to model and forecast the remaining life of assets, prioritize the investigation of asset failures of high consequence, and conduct reliable infrastructure failure risk assessments in support of comprehensive asset management?

Proposed Research

Based upon the key research questions presented above and the known research projects that are ongoing or recently completed by other stakeholders, the following research, demonstrations and technology transfer products are currently being considered by EPA. Each proposal indicates the time frame for the work.

Technology Transfer Product: Optimization of Closed Circuit Television Inspection Data and Information for Effective Condition Assessment – A comprehensive evaluation of the state of the art of CCTV inspection of wastewater collection systems. (6-9 months)

Technology Demonstration Program: Emerging and Innovative Technologies for the Inspection of Wastewater Collection Systems – An inspection technology demonstration program, conducted in cooperation with wastewater utilities. (24-36 months)

Applied Research Review and Evaluation: Understanding the Forensics of Sewer Failures to Support Failure Forecasting Model Development – A comprehensive evaluation of our ability to conduct forensic studies of sewer failures, to understand the critical factors that cause sewer system failures, and our ability to model and forecast future failures to support comprehensive asset management. (12-18 months)

Technology Transfer and Development Program: **Cross-Sector Transfer and Application of Advanced and Remote Sensing Technologies for Wastewater Collection System Monitoring** – A comprehensive review of new, innovative pipeline monitoring technologies that apply advanced and remote sensing approaches. Many of these innovative pipeline monitoring technologies have been developed for application in sectors other than the drinking water and wastewater industries, such as the gas and petroleum pipeline industry. (24-36 months)

Innovative Condition Investigation Method Development: Advanced Techniques for Detecting Exfiltration and Crown Corrosion Conditions – The exploration of using advanced molecular/microbiological techniques for identifying the presence of sulfate-reducing bacteria *(Desulfovibrio desulfuricans)* to assess the probability of crown corrosion in sewers; and using isotope tracers to indicate exfiltration from sewer lines. (18-30 months)

Technology Application Methodology Development: Tools for Inspection Program Prioritization – A tool or tools for assisting wastewater utilities in prioritizing sewer inspections, selecting the appropriate technology and inspection technique, and establishing inspection frequency based on a risk-based methodology. (18-30 months)

System Rehabilitation of Gravity Sewers

The objective of system rehabilitation is to ensure the overall viability of the collection system to maintain operational and structural integrity, and to prevent or reduce infiltration, inflow and exfiltration, and their negative environmental impacts. There are several primary concerns related to deteriorating sewers. Water that flows into sewer pipes through defects (holes, cracks, failed pipe joints) can weaken the critical soil-pipe structure. Fine soil particles carried into the sewer can eventually reduce soil support to a point causing pipe deformation and/or subsidence. Exfiltration of water from the sewer into the surrounding soil can also weaken support provided by the soil. Soil movement due to traffic movement can exceed design assumptions and result in soil-support related problems. Deposition of material and sewer blockages can result in septic conditions due to flat grades, high ambient temperatures and poor ventilation. These conditions are ideal for the development of sulfuric acid and resulting crown corrosion which reduces the structural integrity of the concrete and the reinforcing steel. High rates of infiltration and inflow can lead to sewer overflows, basement backups and excessive peak flows at the treatment plant. Exfiltration of sewage into the surrounding soil can lead to groundwater and soil contamination. In some instances, water pressure drops in water distribution pipes adjacent to exfiltrating sewers can result in contamination of the potable water supply. Lastly, inadequate inspection and quality assurance during sewer installation can result in long-term problems due to poor workmanship. (Tafuri and Selvakumar, 2002)

Generally, rehabilitation includes a broad spectrum of approaches, from repair to replacement that attempt to return the system to near-original condition and performance. Repair techniques are used when the existing sewer is structurally sound, provides acceptable flow capacity, and can serve as the support or host of the repair method. When the existing sewer is severely deteriorated, structurally unsound, or increased flow capacity is needed, it is usually replaced. A wide range of causes can be responsible for sewer line deterioration and failure. These include:

- inadequate or improper bedding material,
- chemical attack,
- traffic loadings,
- soil movements,
- groundwater fluctuations,
- poor design and installation, and
- inadequate maintenance. (WERF, 2000)

Effective system rehabilitation programs require a complete understanding of the condition and performance of the collection system, including factors that affect system integrity and operations. Pipe age is an important factor; however, it is usually a combination of several factors that causes failures and influences rehabilitation decisions, leading to a very complex decision-making process. Pipe materials, bedding and backfill materials, surrounding soil conditions, loads and stresses on the pipe, groundwater levels, sewage and soil acidity, sewage dissolved oxygen levels, and electrical and magnetic fields are factors that negatively impact long-term integrity and operational performance of the collection system. The results from a sound system inspection and condition assessment program provide critical input to decision support tools that evaluate the condition of the system based on structural, hydraulic, service delivery, water quality and economic factors.

Building connections to street sewers, referred to as house or service laterals, can contribute as much as 70 to 80% of the infiltration to a sewer. Fluctuating ground water, variable soil characteristics and conditions, traffic, erosion, washouts, etc., cause enormous stresses on house/service lateral pipes and joints. Connectors and fittings in many cases do not retain their watertight integrity while adjusting to these factors. Some connections react to soil acid and may totally disintegrate in a few years. These conditions often result in generating major points of infiltration at the connection of the house/service lateral to the main. With current technology, rehabilitating building connections may not be economically feasible because of the shear number of connections. The problem is both critical and sensitive because of private ownership and the costs associated with disturbance to the occupant and damage to private property. Because of this, municipalities are often reluctant to address infiltration and inflow problems from these sources. (Tafuri and Selvakumar, 2002)

State of the Technology

Collection system rehabilitation includes a wide range of repair and replacement options that can be used to return the system to acceptable levels of performance. Pipeline rehabilitation procedures are usually preceded by some form of cleaning to remove foreign materials before other phases of rehabilitation are implemented. The removal of roots, sediments, and debris is also a critically important practice for maintaining operational performance levels including ensuring proper flow conditions, reducing infiltration and exfiltration, and preventing structural damage to the pipeline. Common repair methods of chemical and cement grouting address problems associated with groundwater movement, washouts, soil settlements, collapses, and soil voids. Grouting is effective in reducing or eliminating infiltration, but will not significantly improve the structural condition. It does, however, help stabilize the surrounding soil mass. Other system repair approaches include sliplining, spiral-wound pipe, segmented liner pipes, cured-in-place pipe (CIPP), fold and form pipe, close-fit-pipe, coatings, mechanical sealing devices, and spot repair. Many rehabilitation methods reduce pipe cross-sectional area, possibly reducing hydraulic performance.

Trenchless technologies have moved to the forefront sewer system rehabilitation. Many are proprietary systems and the details of installation procedures and materials are trade secrets, limiting the ability to compare and evaluate competing approaches. For some of the technologies, techniques, codes, and standards have been developed; however, because of the rapidly evolving technology in rehabilitation, standards and codes often lag behind. Trenchless technologies are applied in both repair and replacement situations. Pipe replacement technologies include pipe bursting, pipe splitting, pipe reaming, and pipe eating. These trenchless replacement techniques install a new sewer in the location of the old pipe with limited surface disturbance and minimal disruptions to traffic and businesses.

It is significant to mention that past studies have found that rehabilitation of sewers at the street alone does not completely solve the infiltration problem. Successive rainfalls can elevate the groundwater table to levels where entry occurs through these service laterals. When performed, rehabilitation of service laterals is done by point repair or replacement; sliplining and pipe bursting are sometimes used. These approaches do not overcome the private ownership problem or the problems associated with the location and configuration of the line (such as sharp bends in the line).

System rehabilitation includes repairing or replacing appurtenances (manholes, pump stations, wet wells, and siphons) which is an important component of a comprehensive program. About 30 to 50 percent of system infiltration and inflow is due to defects in or near appurtenances, in particular, manholes. For example, manhole covers submerged in one inch of water can allow as much as 75 gallons per minute to enter the system depending on the number and size of holes in the cover. Rehabilitation of manholes, pump stations, and wet wells includes spray-on coatings, spot repairs, structural liners, and replacement. Rehabilitation of siphons includes some of the options available for pipelines such as grouting and lining. Many siphons have been rehabilitated using either cured in place pipe (CIPP) or high density polyethylene (HDPE) liners. (Tafuri and Selvakumar, 2002)

Selection of rehabilitation methods and materials suitable for various parts of the wastewater collection system remains an issue, especially due to the seemingly endless emergence of new materials. Uncertainty in the selection of appropriate repair and replacement techniques is partly related to the lack of understanding of the capabilities of each methodology to solve the problem in the long term. Reliable rehabilitation product performance under actual field conditions, especially over longer periods of performance, is lacking. Data on the effectiveness and longevity of rehabilitation technologies and materials and life-cycle cost information will be useful in determining whether rehabilitation or replacement is more cost effective.

The introduction of sewer pipes made from new plastic materials is challenging the more traditional sewer pipes constructed from concrete, clay and ductile iron. Plastic has, and continues to be an innovative material for sewer pipes. The most commonly used materials for wastewater applications are polyvinyl chloride (PVC), polyethylene (PE) and glass reinforced plastic (GRP). Plastic pipe innovations include structured wall pipe and composite pipe that use different pipe materials to address both structural and corrosion issues. The application of plastic pipe in wastewater is fairly new resulting in the need for determining long-term performance and related testing. In addition, raw materials and formulations can vary widely, resulting in different quality pipe for the same plastic.

Research Questions

The following key research questions relating to gravity sewer system rehabilitation have emerged from the research planning by EPA. These key research questions reflect critical gaps in our knowledge of the performance of innovative rehabilitation technologies, our understanding of the long-term performance and cost of sewer pipe made from new materials, and our ability to determine the most long-term cost effective rehabilitation methods for the situation being addressed.

- Can emerging and innovative sewer system rehabilitation technologies, for both sewer and non-sewer assets, be identified and demonstrated in field settings to improve our understanding of their cost-effectiveness, technical performance and reliability?

- Can approaches and methods be developed for determining the long-term performance and life-cycle cost effectiveness of various system rehabilitation technologies, including new and existing materials?

- Can guidance be provided for establishing a comprehensive system rehabilitation program, including rehabilitation of non-sewer assets, selection of pipe and rehabilitation materials, and testing and quality assurance of field installation and application of rehabilitation technologies?

- Can guidance be provided for collection system operation and maintenance programs, including procedures to assess and optimize maintenance practices that reduce the need for rehabilitation?

- Can sewer and collection system design guidance based on lessons learned from system rehabilitation be developed to enhance long-term performance and system integrity?

- Can a sound, risk-based, decision-making process for selecting optimal system rehabilitation technologies and methods be developed based on long-term effectiveness, system performance, structural integrity, consequence of failure, and life-cycle cost?

Proposed Research

Based upon the key research questions presented above and the known research projects that are ongoing or recently completed by other stakeholders, the following research, demonstrations and technology transfer products are being considered. Each proposal indicates the time frame for the work.

Technology Transfer Products: Collection System Rehabilitation Methods and Technologies – State of the Technology. A series of products that present the current state of the art in the rehabilitation of sewer and non-sewer assets. The first phase of this project will be to conduct an international technology forum to develop a comprehensive inventory of rehabilitation technologies being applied around the world. The second phase of the project will be the development of a series of technology capsule reports that will transfer performance and cost information on rehabilitation technologies using case-studies. (9-36 months)

Technology Demonstration Program: Emerging and Innovative Technologies for Wastewater Collection System Rehabilitation – A rehabilitation technology demonstration program, conducted in cooperation with wastewater utilities (18-36 months)

Technology Transfer Product: **Inspection and Quality Assurance Procedures for Installation and Application of Collection System Rehabilitation Methods and Technologies.** This product will provide technical guidance on the development and implementation of testing and quality assurance practices for field installation of rehabilitation (repair and replacement) of sewer and non-sewer assets. This guide will examine best practices selected from wastewater utilities, vendors and industry. The goal of the product will be to reduce long-term rehabilitation requirements by improving installation practices and collecting critically needed "as built" information for future rehabilitation decision making. (12-24 months)

Technology Transfer Product: Collection System Sewer Pipe Selection Guide. This product will be a comprehensive guide for the selection of sewer pipe for use by wastewater utilities and consulting engineers. This guide will include both traditional and new pipe materials. The selection matrix will present advantages and disadvantages of various pipe materials and pipe designs for use in a wide variety of conditions and applications. (9-18 months)

Applied Research and Review: Collection System Design Based on Lessons Learned from System Rehabilitation. This research effort will look at experience from sewer repair and replacement to determine if approaches for sewer designs can be improved to enhance long-term performance and structural integrity. As collection systems undergo inspection, condition assessment and rehabilitation, data and information that could improve system designs is collected. This effort will review and evaluate those data and information to identify trends and implications on system design. The outcome will be technical guidance on using experiences from rehabilitation to improve site-specific designs as well as general design practices where possible. (12-36 months)

Updated Technology Transfer Product: Design Manual – Odor and Corrosion Control in Sanitary Sewerage Systems and Treatment Plants. This will be an update of the EPA design manual published in 1985 (EPA/625/1-85/018). This updated manual will reflect changes in technologies and practices for controlling odor and corrosion in existing and new collection and treatment systems developed over the last twenty years.

Especially important will be the application of new materials that are designed to be corrosion resistant. (6-18 months)

References

Tafuri, Anthony, N. and Ariamalar Selvakumar, Wastewater Collection System Infrastructure Research Needs in the USA, Urban Water, Vol. 4, Issue 1, 2002.

U.S. Environmental Protection Agency, Handbook – Sewer System Infrastructure Analysis and Rehabilitation, EPA/625/6-91/030, October 1991.

U.S. Government Accountability Office, Water Infrastructure – Comprehensive Asset Management Has Potential to Help Utilities Better Identify Needs and Plan Future Investments, GAO-04-461, March 2004.

Water Environment Research Foundation, An Examination of Innovative Methods Used in the Inspection of Wastewater Systems, 01-CTS-7, 2004.

Water Environment Research Foundation, New Pipes for Old: A Study of Recent Advances in Sewer Pipe Materials and Technology, 97-CTS-3, 2000.

HDD UTILITY TUNNEL TO PEDDOCKS ISLAND - FORT ANDREWS

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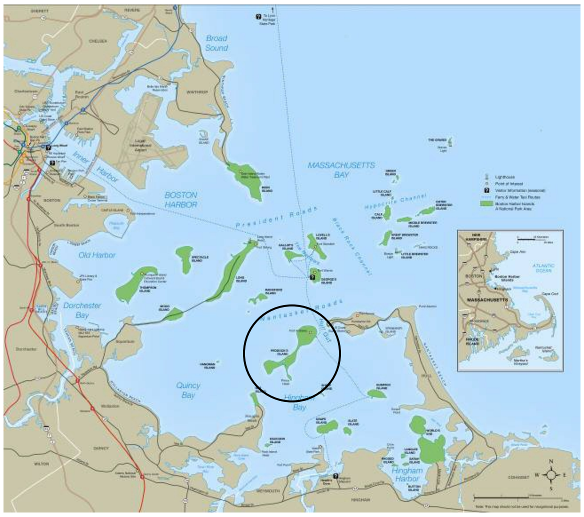
ABSTRACT

Peddocks Island, located between the coasts of Hull and Quincy, Massachusetts, is one of the largest and most ecologically diverse islands in Boston Harbor. Located on the eastern side of the island, Fort Andrews is a magnificent example of Endicott Period military fortifications. However, since 1947, when it was abandoned by the U.S. Army, the fort has fallen into disrepair. Because of its history, Peddocks Island has been certified as an Historical and Archeological Landmark.



Plans are in place for the rebirth of Fort Andrews into a vibrant family camp and eco-retreat that will be the gemstone of the Boston Harbor Islands. However, before the redevelopment of the fort complex could begin, reestablishment of utility service to the island was necessary. The previously existing 100-year old submarine water and electric services to the fort had been destroyed, and no water or electric service to the island existed, except for a small photovoltaic system. An aged sewer system that discharged directly to the ocean was also unusable.

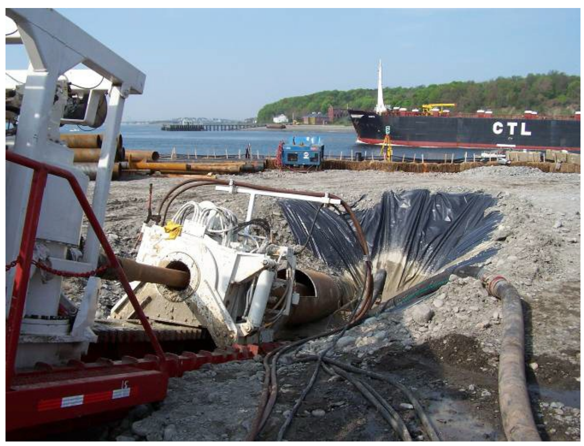
A feasibility study of alternatives to bring utility service to the island was conducted, with the selected alternative consisting of horizontal directional drilling approximately 2,000 linear feet beneath Hull Gut, a major shipping channel for Boston Harbor, from the Hull main land to Peddocks Island. A single 20-inch diameter steel casing within the drilled bore hole housed HDPE conduits that provided power, communications, wastewater and water service to the fort complex to fullfill the projected utility needs for the planned island buildout.

INTRODUCTION

Peddocks Island, located between the coasts of Hull and Quincy, Massachusetts, is one of the largest and most ecologically diverse islands in Boston Harbor and is a key component of the Boston Harbor Islands National Park Area. Fort Andrews, located on the eastern end of the Peddocks Island, is a magnificent example of Endicott Period military fortifications, with most of its brick buildings and gun emplacements constructed between 1897 and 1918. The fort was highly active through World War II, with dozens of buildings housing active military personnel and prisoners of war. Because of its history, Peddocks Island has been certified as a Historical and Archeological Landmark.

When Congress established the Boston Harbor Islands as a unit of the National Park System in 1996, the Boston Harbor Island Partnership was created to coordinate the activities of the island owners and managers, both public and private. The Island Alliance, a private nonprofit organization, holds one of thirteen seats on the Partnership and is charged with raising private funds to support the park and to serve as a catalyst, collaborator, and business planner to ensure implementation of the overall Economic Sustainability Strategy. Other Partnership members include the National Park Service (NPS) and the Urban Parks Division of the Massachusetts Department of Conservation and Recreation (DCR), which owns the island.

Island Alliance is undertaking the restoration of the Fort Andrews portion of Peddocks Island to create a unique destination for Boston-area residents and visitors to the island. The vision for the island includes the development of an eco-retreat and family camp which will provide a new day-use and overnight destination for island visitors. It is hoped that this development will provide a new revenue stream to help support the Boston Harbor National Park Area, preserve the former military fort on Peddocks Island, and attract new visitors to this unique and convenient destination.



However, before the redevelopment of the fort complex could begin, reestablishment of utility service to the island was necessary. The 100-year old submarine water and electric services to the fort had been destroyed by storms and channel dredging activities, and no water or electric service to the island existed, except for a small photovoltaic system. An aged sewer system that discharged directly to the ocean was also unusable.

In 2003 the Island Alliance, with its project manager Jones Lang LaSalle, engaged **Environmental Partners Group** of Quincy, MA to prepare a feasibility study for restoration of utility services to the island. Environmental Partners was subsequently engaged to provide full design and construction-related services. Power Engineers of Shrewsbury, MA assisted Environmental Partners on the electrical aspects of design and construction. The project was formally designated as the *Peddocks Island-Fort Andrews Preservation and Adaptive Re-Use Project.*

UTILITY NEEDS

The scale of the Peddocks Island-Fort Andrews Preservation and Adaptive Re-Use Project is substantial. Visitor numbers, water/sewer flows, and power needs are projected to grow steadily, and by the year 2020 nearly 1,400 day and overnight visitors are projected to use the island on a daily basis during the summer season. These visitors are projected to consume nearly 45,000 GPD of water and generate as much as 30,000 GPD of wastewater. The island could also ultimately consume over 400 kilowatts of power per day.

Major utilities need to be simple to operate and maintain, flexible for growth, highly reliable, and easy to transition between periods of low use and periods of extreme use. Also, given the island setting, it would be cost-prohibitive to scale up the utilities over time, especially if some or all of the utilities were brought over from the mainland.

FEASIBILITY STUDY FINDINGS

The significant findings and conclusions of the feasibility study with respect to the major utility needs were as follows:

- The glacial till soils on Peddocks Island are poorly suited for septic systems. Furthermore, ocean disposal of treated wastewater was not considered to be a viable option.
- The Hull wastewater collection/treatment system had available capacity and is located one half-mile from Peddocks Island on the east side of the Hull Gut shipping channel.
- There is no viable, on-island source of water for use in the redevelopment plan. The only viable water supply options were reconnection of a 100-year old, 6-inch water main across the two-mile length of the island to Quincy, or construction of a new supply line to the public water system in Hull. Either route would require a channel crossing to the mainland.

- With an appropriate rehabilitation program, the existing water and sewer systems within the fort area could be largely re-used, thereby reducing program costs and disturbance of the fort grounds.

- Only a power connection to Hull provided the reliability and flexibility necessary for the project at a reasonable cost, while still allowing for potential renewable energy sources (wind turbines) to be employed on the island, with direct connection to the power grid.

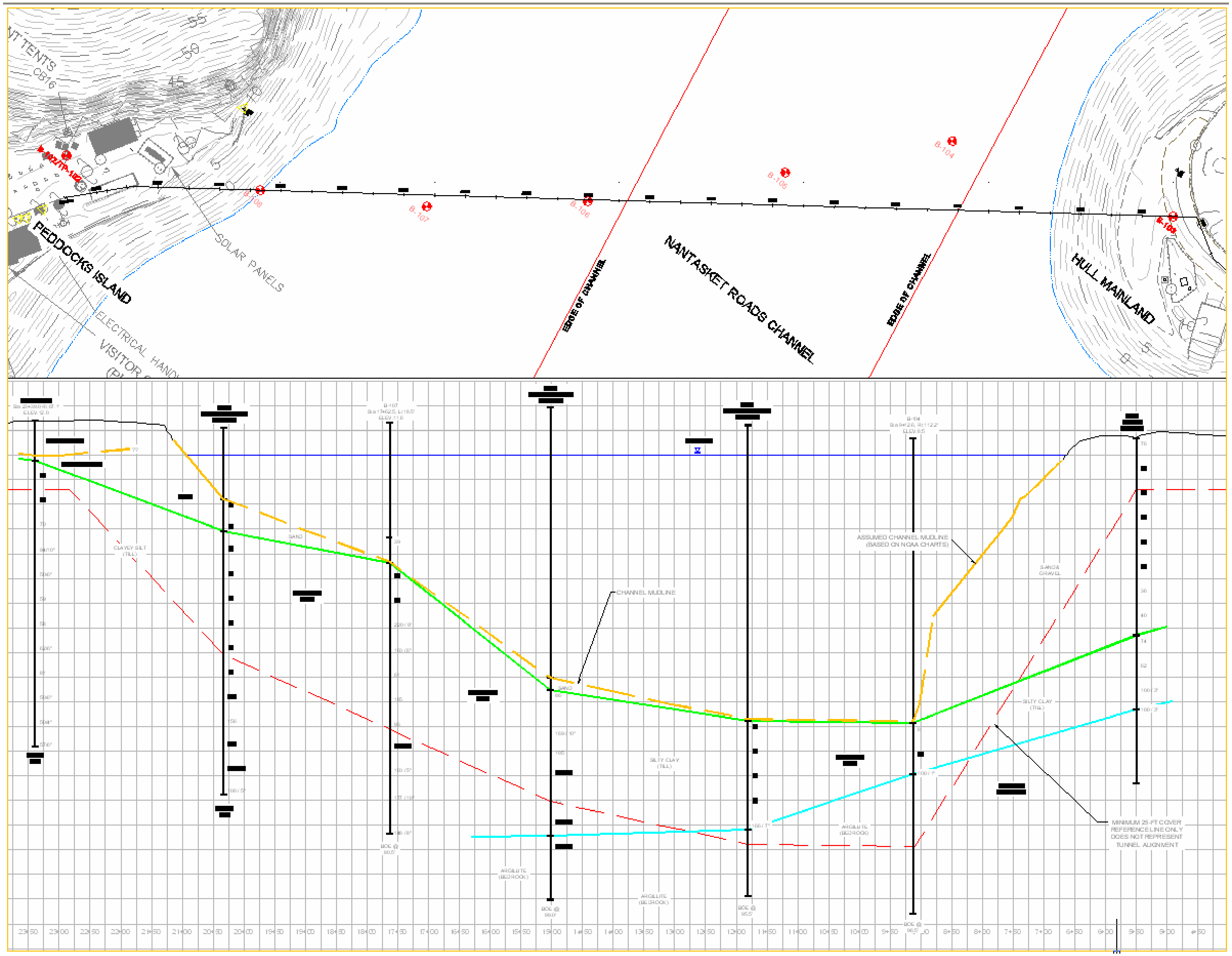
- Given the relatively close proximity of all three major utilities on the Hull mainland, Environmental Partners recommended that submarine water, sewer, power and communication connections be made across Hull Gut. Accomplishing the Hull Gut Channel crossing could be done via three basic methods: direct placement on the bottom; cross-channel dredging and burial; and horizontal directional drilling (HDD).

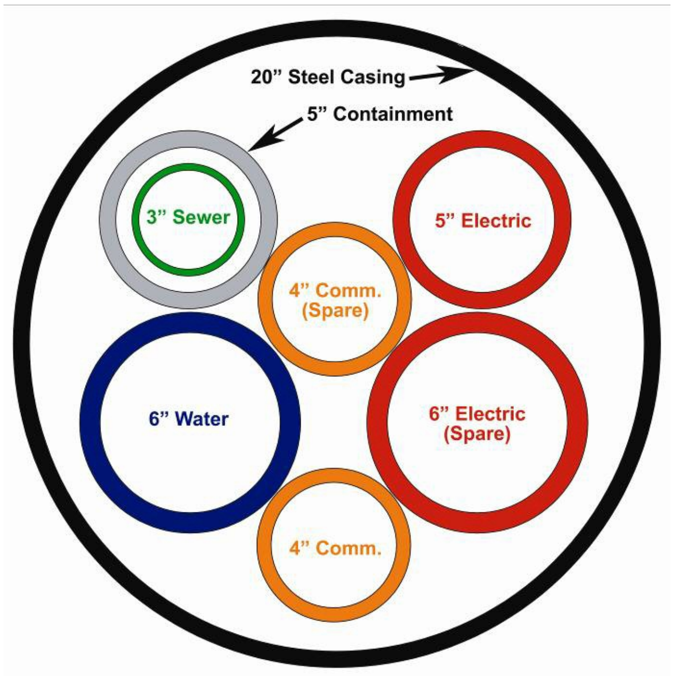
- Since Hull Gut is an active shipping channel that is periodically dredged, only HDD was deemed to be a truly secure approach over the long-term. HDD was the only viable method that would provide the 25-foot minimum separation buffer between the channel bottom and the utility crossing that was required by the U.S. Army Corps of Engineers to protect the utilities during future dredging of the Gut.

- The water, sewer, electric and communications utilities would then be bundled and installed within the tunnel. Once the utility tunnel was complete, there would be no utility-related impediments to fulfilling full-scale redevelopment plans for Peddocks Island

DESIGN ELEMENTS

Environmental Partners began design of the utility tunnel and connections in October 2004. The first significant design task was to complete a preliminary tunnel layout plan and gather geotechnical information on the channel crossing route. A jack-up barge was deployed to Hull Gut in mid-December 2004, at which time five deep borings were advanced across the 1,500-foot wide channel. The site stratigraphy consists of sand, gravel and some till over argillite bedrock on the mainland, transitioning into nearly bare bedrock within the channel.


PEDDOCKS ISLAND
SOLAR PANELS
ELECTRICAL
VISITOR
EDGE OF CHANNEL
NANTASKET ROADS CHANNEL
EDGE OF CHANNEL
HULL MAINLAND
CHANNEL MUDLINE
ASSUMED CHANNEL MUDLINE (BASED ON NOAA CHARTS)
CLAYEY SILT (TILL)
SAND
SILTY CLAY (TILL)
SAND & GRAVEL
ARGILLITE (BEDROCK)
MINIMUM 25-FT COVER REFERENCE LINE ONLY DOES NOT REPRESENT TUNNEL ALIGNMENT

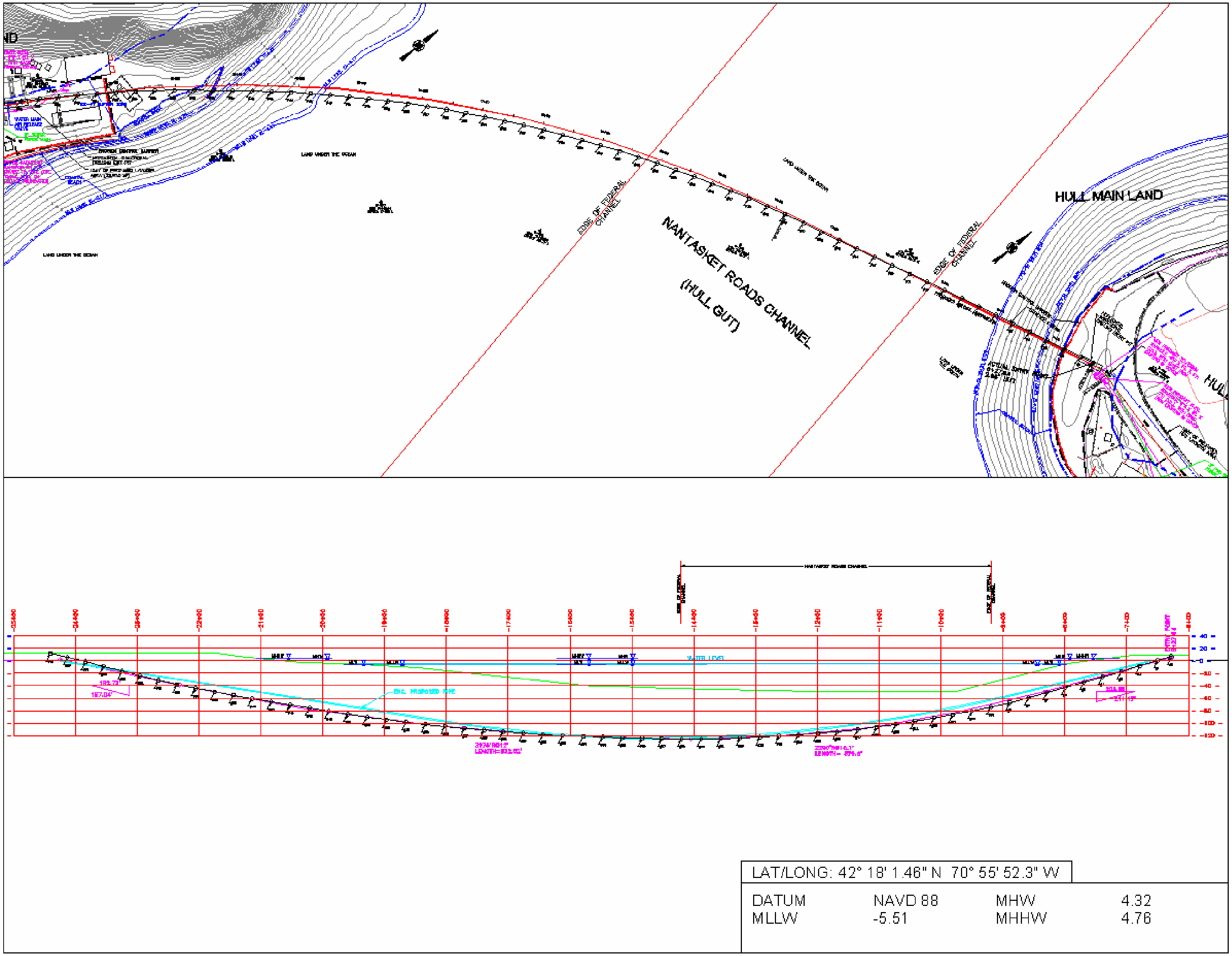


In order to control project costs while providing for the island-specific utility needs described earlier, Environmental Partners selected a single barrel tunnel design consisting of a 20-inch steel casing. Within the steel casing a bundle of high density polyethylene (HDPE) pipes would be installed, including a 6-inch water main, a double walled sewer force main (3" ID), primary power and communications conduits, and a pair of spare conduits. The double-walled sewer piping was a negotiated compromise approved by the Department of Environmental Protection, which allowed the inclusion of both the water main and sewer force main within a single HDD borehole.

The steel casing was not intended to provide any structural strength to the conduit bundle, but served as a smooth sleeve for pulling the complete bundle through the potentially jagged bedrock walls of the borehole. As the conduits were pulled through the casing, low strength flowable slurry was pumped into the casing to fill the annular space. High strength slurry was pumped into the leading edge of the casing at both the entry and exit pits to plug the ends of the casing.

The tunnel alignment and construction approach was selected in recognition of the limited working area available on the mainland, the depth of the shipping channel within Hull Gut, and the spatial needs for staging and fabricating 2,000 feet of steel casing and utility pipes. The vertical curve of the tunnel would take it down to a maximum depth of 125 feet below sea level.

The HDD drilling equipment would be set up on the mainland, with its back-end located only 15 feet from the fence enclosing the Hull High School baseball field. While drilling operations were underway, the steel casing would be fabricated on the island in two 1,000-foot long pieces, which would be welded together during the pull-back. The HDPE utility pipes would be butt-fused on the island in continuous 2,000-foot lengths. Given their greater flexibility, the HDPE pipes could be snaked along one of the islands foot paths beyond the end-point of the steel casing. Connection points for all of the utility pipes, including a duplex wastewater pump station, electrical switchgear, and water mains, were located within several hundred feet of the tunnel end points on the island and mainland.


HULL MAIN LAND
NANTASKET ROADS CHANNEL
(HULL GUT)
EDGE OF FEDERAL CHANNEL
EDGE OF FEDERAL CHANNEL
LAT/LONG: 42° 18' 1.46" N 70° 55' 52.3" W
DATUM NAVD 88 MHW 4.32
MLLW -5.51 MHHW 4.76

CONSTRUCTION PHASE

Construction began in May 2006. J.F. White of Framingham, MA was selected as the general contractor. The HDD subcontractor was Michels Directional Crossings of Brownsville, Wisconsin. Elements of the Hull Municipal Light Plant provided switch gear and made electrical and communications connections on the mainland side of the tunnel. The overall project contract price was $3,646,500. All aspects of construction progressed smoothly, on time, and under budget.

The tunnel construction began on May 8. Due to the relatively loose soils encountered on the Hull side of the Gut, a 36-inch diameter steel surface casing was initially driven approximately 280 feet to bedrock to ensure that the borehole remained open during the drilling operations. Michels augered the loose soils from within the surface casing and then used a 12-inch, 3-cone roller bit to drill the pilot hole to the exit pit on Peddocks Island. A single reaming pass was made with a 30-inch, 5-cone roller bit. After the reaming pass was made, pull-back of the steel casing was completed in one day, and the HDPE utility pipes were pulled-back during the following day. Michels demobilized from the site on June 27. Thereafter, J.F. White made the necessary utility connections at both ends of the tunnel and the project was substantially completed in mid-October 2006.

To monitor the location of the pilot bore, Michels relied on a Tru-Tracker system. However, because Hull Gut is an active shipping channel, the United States Coast Guard would not allow Michels to station their monitoring equipment beyond 100-feet from either the Hull or Peddocks Island Shoreline. Therefore, actual tracking of the pilot bore was limited to the area within 100 feet of either shoreline. Within the Gut, Michels relied solely on positioning calculations to determine the location of the pilot bore. Even with these limitations, the pilot hole was completed in the first attempt and exited on Peddocks Island within two feet of the intended target.

LESSONS LEARNED/FUTURE PROJECT CONSIDERATIONS

With any project, hindsight can provide a vision of differing approaches that could have increased the overall efficiency of the project. Hindsight can also emphasize areas where additional attention could be spent during future, similar projects that could also improve project efficiency.

For the Peddocks Island project, the unique setting provided challenges that tested the design team and contractor’s ability to adapt to the project’s physical constraints. For future projects, issues to keep in mind include:

- Consideration of the space requirements for the assembly of the casing and utility piping. For maximum efficiency, the full length of casing and utility piping should be assembled into single lengths so that they can be pulled non-stop through the drilled hole. If suitable length is not available to assemble the complete length, the amount of stops necessary to join the sections should be kept to a minimum.
- Consideration of the space requirements for the equipment at the entry pit. For the Peddocks Project, the space limitations required that the contractor reconfigure their drilling equipment perpendicular to the entry pit.
- Because of the significant currents through Hull Gut, the typical floating barge used to locate the drill bit could not be used, and a spud barge was deployed to the site to provide a stable base for the survey equipment. It is important to consider the project survey logistics prior to initiating the project.
- The importance of collecting subsurface information before beginning any HDD project can not be over-emphasized. Having as clear a picture as possible of the subsurface conditions will minimize the potential for change orders. Because of the extensive subsurface investigations conducted prior to the design and bidding phases, the design team and the bidding contractors had a clear picture of the conditions that would be encountered during directional drilling. This up-front knowledge was an important factor in ensuring that change orders would not be required by the selected contractor.

PROJECT CONCLUSIONS

- This project represents the use of ingenuity and design of a unique approach, wherein multiple utilities, including water, sewer and power were installed within the same HDD tunnel structure to serve an island location. Separate tunnels or cut-and cover channel crossing techniques would have been too costly or vulnerable to damage within the channel, thus threatening the fundamental viability of the Island Alliance’s vision. With utilities now restored to the island, the Island Alliance can advance restoration and rehabilitation components of the master plan, and accommodate more day and overnight visitors immediately.
- This project will allow the realization of the larger vision for Peddocks Island, that is, to develop and manage the island in a sustainable manner, while also generating resources to protect and preserve its unique historic and environmental character base. The residents of the metro Boston area will be able to visit and

enjoy, for extended stays, an island environment and historic fort complex that would otherwise be largely inaccessible.

- The Island Alliance, the Department of Conservation and Recreation, and the other eleven members of the Boston Harbor Islands Partnership have closely followed this project and were justifiably concerned over potential cost overruns and the technical complexity of restoring permanent utility connections to support their ambitious plans. There is now a recharged sense of excitement and anticipation for the next stages of development to get underway, and a confidence that sound engineering judgment can overcome virtually any obstacle they encounter.
- Complex elements of the project included highly limited staging and work space for the HDD equipment on the mainland, the need to work in limited space among historic buildings on the island, and the limited ability to track HDD progress while under the shipping channel.
- The base project was completed two months ahead of schedule, free of change-orders, and $20,000 below the base contract price of $3,646,500. With this success, in hand, the Island Alliance used contingency funds to make additional utility improvements to the fort's old guardhouse/new visitor center.

Design and Construction of a Large Diameter Welded Steel Pipe Bridge Crossing for Potable Water Supply in Anchorage, Alaska

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Introduction. Engineering design and construction can be especially challenging in Alaska because of the harsh, remote, and sensitive environment. This paper focuses on the design and construction challenges of creek crossings of a large diameter potable water transmission main in the Municipality of Anchorage (MOA), in Southcentral Alaska.

Project Description. In order to meet the growing long term potable water supply demands, several large diameter pipeline water supply projects have been undertaken by the Anchorage Water and Wastewater Utility (AWWU) over the last two decades to bring water from Eklutna Lake in the Chugach Mountains in the MOA to the Anchorage Bowl in the MOA, a distance of approximately 30 miles.

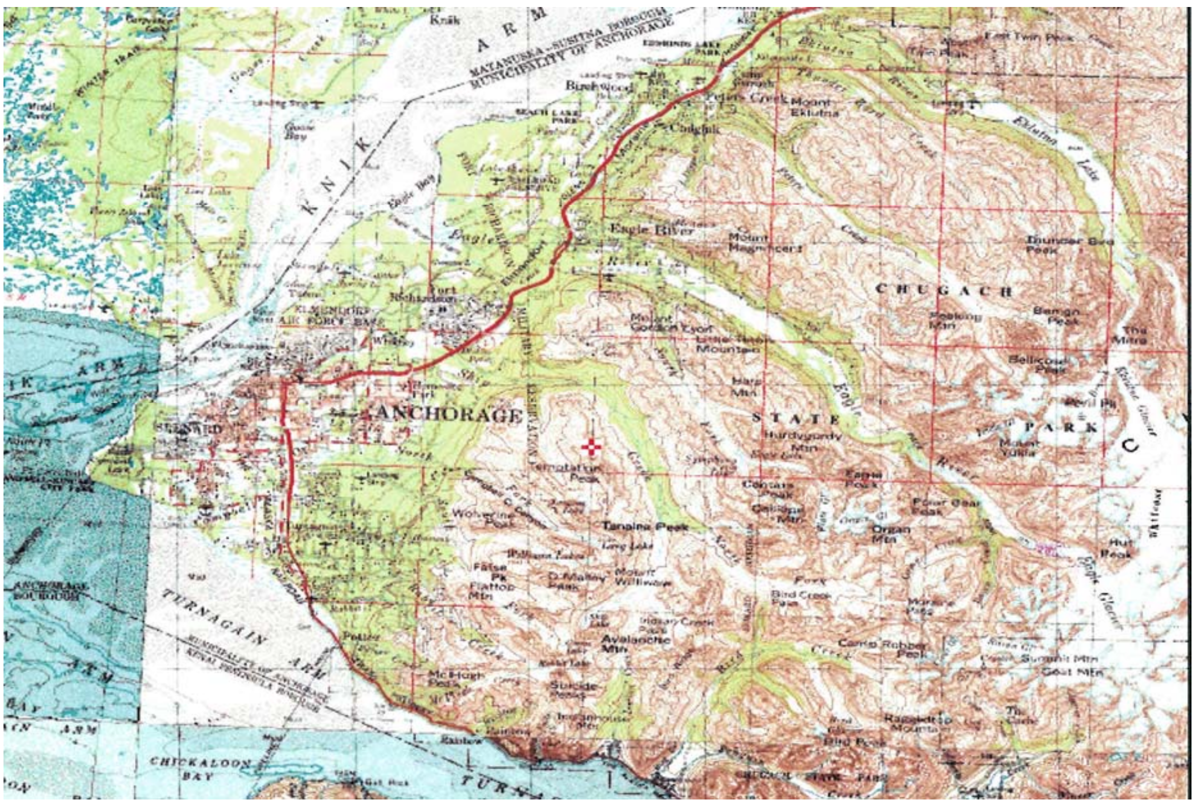

Figure 1 Eklutna Lake and the Anchorage Bowl

The Anchorage Loop Water Transmission Main Project carries water from northeast Anchorage's Ship Creek Water Treatment Facility, where the "Eklutna Line" ends south through East Anchorage to supply the Hillside, then west to serve thousands of consumers in southwest Anchorage.

The Anchorage Loop Water Transmission Main, Phase IV, (Loop Phase IV) is the last segment of this eight-phase project extending a distance of approximately 23,200 feet through southeast Anchorage. With water flowing from northeast to southwest, Phase IV begins at the Tudor Reservoir Tanks and follows Tudor Road west to Boniface Street/48th Avenue, where it turns south and west and until it intersects with Bragaw Street. At Bragaw, it runs south and follows the Bragaw/Abbott Loop Extension corridor to near East 84th Street. The pipeline diameter is 48-inches for the eastern 10,700 feet, and reduces to 42-inches for the remaining 12,500 feet of the project.

Loop Phase IV was further fragmented into four separate construction projects to join with the State and local agencies in their two road building efforts along the corridor.

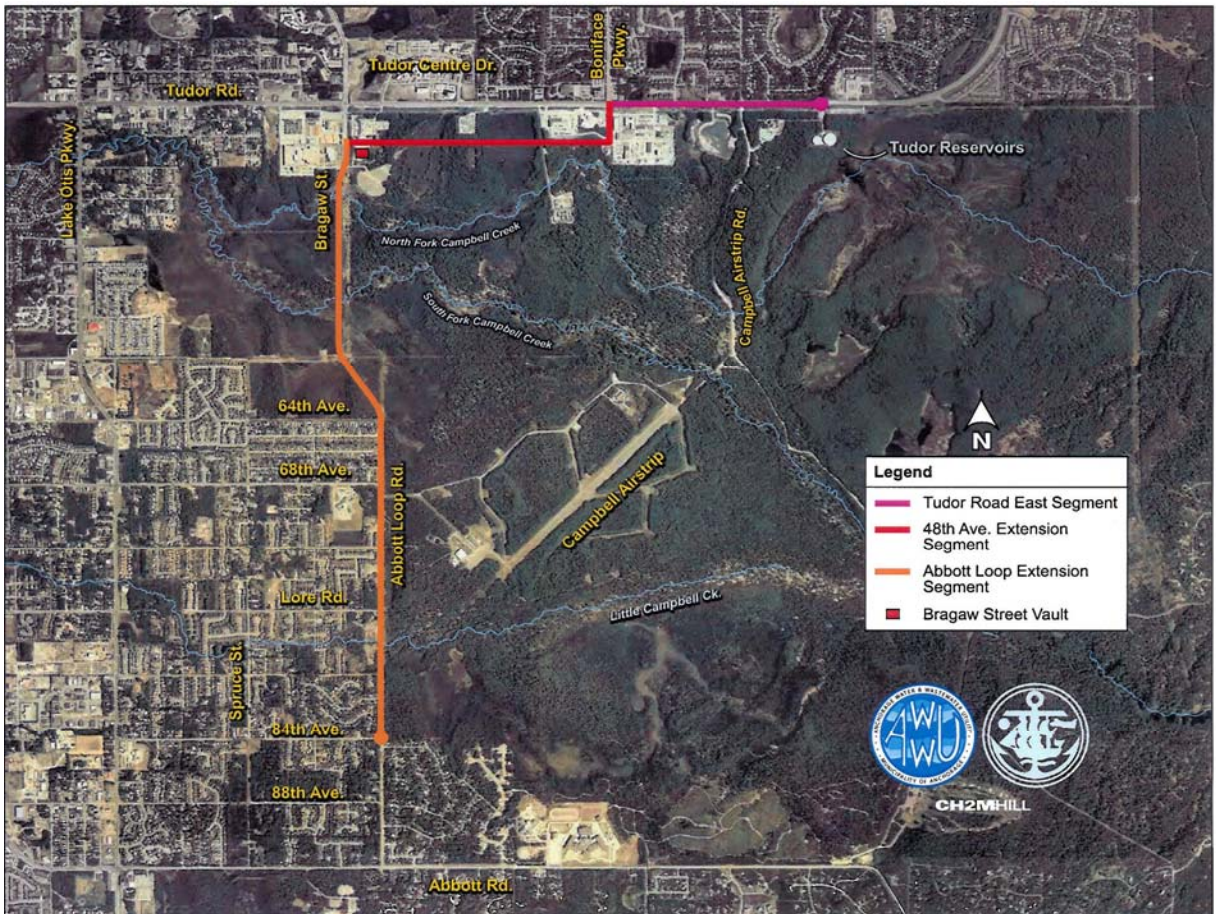

Figure 2 Loop Phase IV Project Overview

This subject project is the first of the Loop Phase IV projects, and is known as the "Loop Phase IV – Abbott Loop Extension (ALE). " The ALE project consists of 12,500 feet of high pressure (over 200 psi), 42-inch diameter, welded steel pipe. The majority of the pipe is buried with a minimum cover of 8 feet for freeze protection; and the region is seismically active.

Project Design. From project start to finish, the ground slopes from elevation (EL) 232 at the south end of the project to EL 158 at the northeast end of the project. Groundwater elevations vary throughout the project from at the surface to below the pipe trench. There are two areas of distinctly different soils along the project corridor. One area is composed of peat (depths of 7 feet) overlying dense outwash (gravel and sand with cobbles and boulders) and alluvial deposits (gravels and sands). The other area is composed of glacialfluvial deposits (sand, gravel, cobbles and boulders with a fine-grained matrix of silt).

The pipeline project was included with a large road construction project. The road project consisted of a new road through an undeveloped area and the reconstruction of an existing two-lane rural road. Construction of the ALE Project provided a particular challenge as it involved work in an area that provides black bear, brown bear, and moose habitat; and three creek crossings that provide Pacific salmon habitat within the MOA. In addition to four lanes of new roadway, the new road included construction of two new bridges over the North and South Forks of Campbell Creek and the Lore Tributary to the South Fork.

Because of the threat of buried glacial erratics, lack of local contracting experience, limited right of way (ROW) in which to build a vertical shaft, and high cost, micro-tunnel boring machine (mtbm) construction was not considered a feasible option. For environmental reasons open trench excavation through the creeks was not an option accepted by various resource agencies. Crossing the creeks under the road bridge spans was determined the only acceptable alternative for the pipeline path. Thus, the bridges feature Large Diameter Welded Steel Pipe Bridge Crossings that are the subject of this paper.

The North Fork of Campbell Creek consisted of two 360-foot, 42-inch welded steel pipe crossings on the east and west side of the bridge. The East side pipe was a replacement of an existing 30-inch pipe buried beneath the creek, and the West side pipe is the Loop IV pipe.

The South Fork of Campbell Creek and Lore Tributary consisted of one 568-foot, 42-inch welded steel pipe crossing on the west side of the bridge.

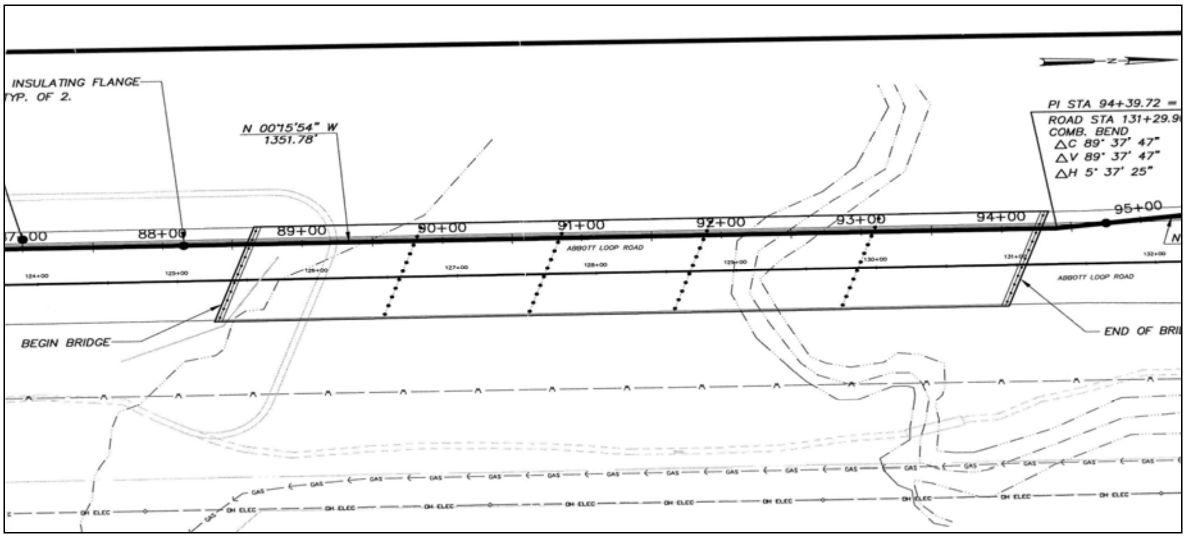

Figure 3 South Bridge and Pipe Plan

Bridge Design Criteria. Criteria for design of the roadway bridges are given in *Abbott Loop Extension – 48th Avenue to Abbott Road, Draft Design Study Report*, April 2004. These criteria were reviewed to develop comparable criteria for the pipelines. Bridge design criteria of interest are seismic and thermal.

Bridge seismic and thermal criteria include:

1. Acceleration Coefficient "A" will be equal to 0.45.
2. Soil Profile Type I (Site Coefficient, S = 1.0).
3. Importance Classification, Essential bridge (IC = I).
4. Seismic Performance Classification, SPC = D.
5. Lateral movement of bridge struct. relative to adjacent approach fill = 3.4".
6. The base mean temperature (installation temperature) is 50 degrees F.
7. Temperature range is –50 degrees F to +100 degrees F.

Pipe on Bridge Design Criteria. Seismic and thermal criteria were selected to follow bridge design criteria. Other criteria included for pipe internal pressure and thrust restraint.

Additional Pipeline thermal criteria are:

1. The base mean temperature (installation temperature) is 50 degrees F.
2. Temp. range is +32 degrees F to 70 degrees F when pipe is full of water.
3. Temp. range is –50 degrees F to +100 degrees F when pipe is empty.
4. Resulting pipe thermal contraction/expansion, from 50°F, will be:

Table 1 – Pipe Thermal Performance

	Full Pipe		Empty Pipe	
Location	Service Temp 32°F	Service Temp 70°F	Service Temp -50°F	Service Temp 100°F
360′ N Bridge	-0.51″	+0.56″	-2.81″	+1.40″
568′ S Bridge	-0.80″	+0.89″	-4.43″	+2.22″

Pipeline pressure & thrust criteria:

1. Operating hydraulic grade line (HGL) ranges from 479′ to 630′. Corresponding pressures at elevation 185′ will range from 127 to 193 psi.
2. Test HGL is 682′; corresponding test pressure at elevation 185′ is 215 psi.
3. Pipeline thrust loads on bridge structures are limited to the extent possible.
4. Pipeline fixed to the bridge is able to flex with bridge movements (camber, dead loads of pavement, pipe and water, live loads, and long-term creep), creating no additional load on bridge or pipeline. Major bridge flexures are approximately:

Table 2 – Anticipated Bridge Flexures

Load Condition	Weight/LF	Mid-span Vertical Δ	20′ Jt Defl	40′ Jt Defl
T-beam only	-----	+1.35″	-----	-----
4″ AC on T-beam	350	0 = planned curve	0.15 deg	0.30 deg
Empty 3/8″ wall pipe*	87	-0.34″	< 0.15 deg	<0.30 deg
Water in pipe*	301	-1.16″	< 0.15 deg	<0.30 deg
Live load	HS25	-1.22″	< 0.15 deg	<0.30 deg

*Pipe and water weight is distributed between two adjacent T-beams

Other pipe criteria:

1. Pipe and supports should not extend below bottom edge of girders.
2. Minimize pipeline construction impact on ground surface beneath bridge.

Pipe Support on Bridges. Alternative pipe materials - ductile iron pipe (DIP), welded steep pipe (WSP), and AWWA C303 concrete cylinder pipe (CCP) – were specified for buried sections of the pipeline, so options using these materials were examined for pipe mounted on the bridges. Table 3 summarizes these options.

Table 3

ISSUE / OPTION	A. DIP, Supports at 20' Intervals	B. WSP, Supports at 20' Intervals	C. WSP,120' Span Between Piers	D. CCP, Supports at 20' Intervals
1. Pipe	20' DI, Class 250, Restrained Joint	40' WS, wall t 3/8", plain ends & cplgs	Continuous WS, wall t 3/4", buttweld	If Concrete Cylinder
Pipe Lbs/LinFt	243	174	343	400+
Coating/Insulation	76	76	76	76
Water Lbs/LinFt	644	601	601	601
Total Lbs/LinFt	963	851	1020	1076+
2. Pipe support on bridge	Adjustable saddles @ 20' on ctrs, behind bells, w/ teflon slides, no pipe sag between supports	Adjustable saddles @ 20' on ctrs, 10' from pipe ends, w/ teflon slides, no pipe sag between supports	Ring girders only at abutments & piers, no supports/weight on T-Beams, continuous-beam spans sag <4"	Similar to option B, requires analysis of thin cylinder wall for saddle loads, etc
3. Pipe joint flexibility for bridge camber & flex, also seismic movement	Provided w/ (4), RJ can deflect 0.5 degree after assembly. Possible joint issues.	Provided w/ (4), Depend-O-Loc can flex 1.5 degrees after assy. Possible joint issues.	Pipe supported independently of bridge structure, no joint issues	May require pipe wall t greater than normal CCP cylinder

Options A and B, based on DIP and WSP, were developed using saddle supports at 20′ intervals. Option C, based on WSP, was developed using butt-welded joints to span 120′ from pier to pier. Option D, based on CCP, was found to be heavier and not suited for saddle support; it would have to be designed as WSP. No cost estimates were made to compare the options, but a general review including constructability indicated overall cost to be similar between Options A, B, and C.

It was decided that Option C, self-supporting WSP spanning from pier to pier, would be easiest to construct and most reliable, and it was therefore the recommended choice for design.

Pipe Bridge Design. The continuous 5-span pipe bridge over the South Fork of Campbell Creek is a self-supporting pressurized pipeline designed for 200 psi working pressure and comprised of five-120 feet long spans. The pipe wall was designed to limit pipe gravity bending stress from beam action (dead load of pipe and insulation combined with live load from water) to approximately 13 ksi which limited deflection to approximately 1.5″ maximum. This required heavy walled (42″x 3/4″) spiral welded steel pipe with complete joint penetration welded butt joints and 100% x-ray inspection.

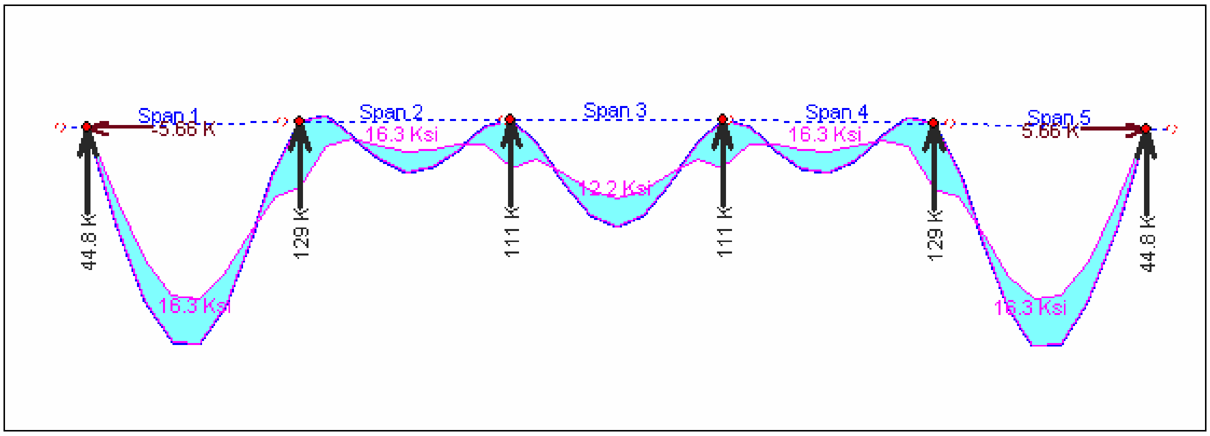

Figure 4 – Pipe Bending Stress Analysis

Accommodation of Transverse Movement Behind Abutments. Behind each abutment, for both bridges, two flexible ball joints on a 20 foot vertical section of pipe were designed to allow for the thermal and seismic induced transverse movement of the pipeline and approach fill relative to the bridge structure. The ball joints (available in DI and welded steel) allow up to 15 degrees of axial rotation and permit 4-8 inches of piping movement in the longitudinal direction at each end of the pipe bridges. No capacity for longitudinal extension was designed for this flexible ball joint system, as the welded joint pipe beyond the ball joint assembly accommodates the remaining transverse movement.

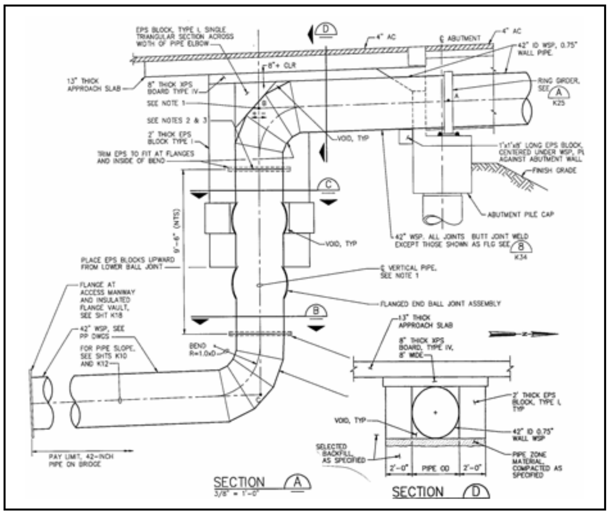

Figure 5 Ball Joint Assembly

Pipe Transitions off Bridges. Three alternatives for transitioning the pipe from the bridges to underground were evaluated. The first alternative, expansion loops in vaults, was found to require large, complex structures. The second alternative, using a simple vault housing an expansion joint, required an extended casing to permit longitudinal pipe movement between the bridge and the vault. The third alternative, using articulating joints, required only a small vault and was the recommended alternative.

Air Release from Pipe on Bridges. The pipeline profile follows that of the bridge, and requires a combination air release valve (CARV) at or near the high point in the pipeline profile for three purposes – to expel large quantities of air when the pipeline is filled with water, to admit large quantities of air when the pipeline is deliberately or inadvertently drained, and to expel small quantities of air which accumulate during normal operation of the pipeline. The profiles of pipelines on the bridges were examined to identify the optimum location for air valves. Only one air valve was required for each pipeline on each bridge. For both the North and South Fork bridges, the air valves were located at Pier 3. The ring girder supporting the pipeline is fixed at Pier 3 to eliminate longitudinal movement of the 42" pipe relative to the air pipe(Because of the skew of the bridge relative to the pile caps and the slope on the pile caps, Pier 4 was fixed with the air valves on one of the spans of the North Bridge).

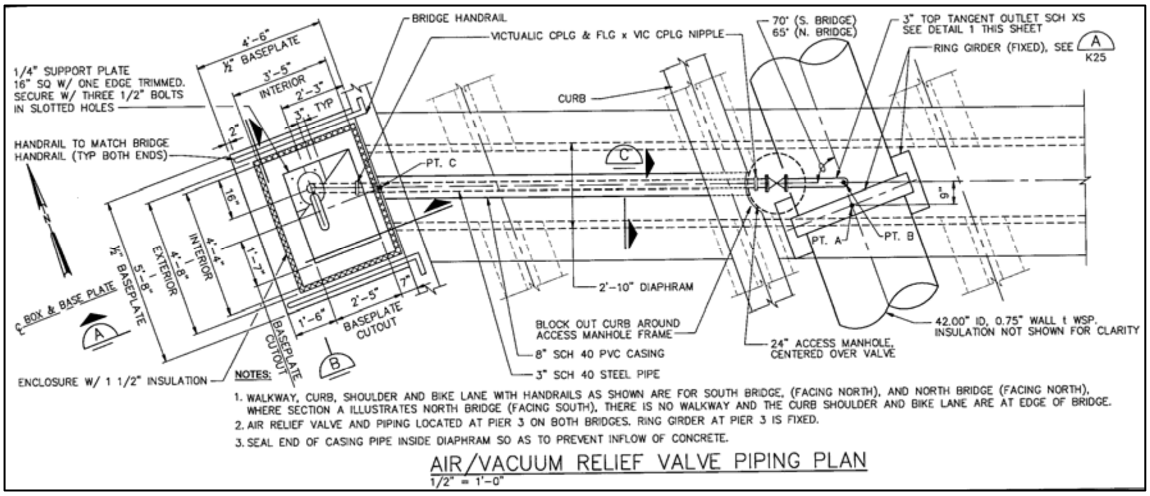

Figure 6 Bridge Air Release Valve

The air valves were located at piers to route connecting piping through the diaphragm from the 42-inch main to the air valve located at the edge of the bridge. Pier 3 (and 4 in one instance) is closest to the actual high point in each pipeline, leaving a small air pocket at the top of each pipeline 1.4″ deep on the North Bridge, and 1.9″ deep on the South Bridge. This small air pocket reduces the cross-sectional area of the water column by about 1 percent, having negligible effect on carrying capacity of the pipeline, and otherwise not interfering with normal pipeline operation. It was necessary to expel this air pocket before the pipeline was hydrostatically tested to avoid compromising the test (air dissolves into solution causing test pressure to drop).

While 6″ CARV's were used at other high points along the Abbott Loop pipeline, smaller valves were used on the bridges. Space constraints within the diaphragm limited air valve piping to 3″ diameter. The 3″ connecting pipe was heat traced and insulated, and was installed inside an 8″ casing to allow removal and replacement. The CARV was installed in a vandal-resistant fiberglass insulated enclosure at the edge of the bridge, and was heat traced and insulated. The enclosure was not heated.

Isolation Valve to Allow Draining of Pipe on Bridges. Consideration was given to the potential need to drain the bridge-mounted pipelines to prevent freeze-up during a prolonged zero-flow period or to facilitate repair of pipe on the bridges. Review of the pipeline profile from the line valve station at 88th Avenue and Abbott Loop Rd to the line valve station at Bragaw Street and Tudor Road indicated that a single valve near the south bridge was necessary for this purpose. It was located south of the blowoff south of the bridge, at Station 81+00. It prevents water from draining out of higher sections of the pipe from the south, and limits drainage to the volume contained in the pipe on the bridges. This limited volume can be removed from the blowoffs south and north of the south bridge and at the valve vault at

Bragaw and Tudor. A buried vault for a full-line size 42-inch butterfly valve was recommended.

Pipeline Insulation. Because the pipeline on the bridges is exposed to atmosphere, protection from freezing was an important design consideration for the above-grade water pipeline exposed to freezing temperatures. A heat loss analysis was conducted to determine the appropriate insulation thickness for these pipeline sections on the bridge.

The worst case scenario for pipeline freezing is a zero-flow condition on the coldest winter day. Heat loss calculations for occupied structures commonly use climatic design information presented in ASHRAE design publications. 2001 ASHRAE Fundamentals, Chapter 27, Table 1A presents the 99.6% heating dry bulb value (Anchorage, AK – Ft. Richardson) as –19°F. Local observations suggest that actual temperatures in the area are colder on average than surrounding areas. Extreme cold days certainly drop below –19°F, and may approach –35°F on occasion. Accordingly, the –50°F design temperature was the conservative value used for this analysis.

The pipeline section subject to analysis was welded steel, 42-inch nominal diameter, with ¾-inch wall thickness. The analyzed pipeline has exterior insulation covered by an aluminum jacket for weather and damage resistance. Incoming water temperature can be as low as 36°F, and that value was used in the calculations.

The minimum allowable water temperature in the pipeline analysis was 32°F, the onset of freezing. The analysis calculated the time it took, under a zero-flow condition, for the water temperature to drop from 36°F to 32°F within the pipeline.

The analysis used a commercially available cellular-glass block insulation product, which is produced in several configurations to match any pipeline diameter. Thermal conductivity in the expected temperature range is approximately k=0.25 Btu-in/(hr-ft^2-°F). Surface conductance losses become significant on cold, windy days. Based on local observations and design practice, a 15 MPH wind speed was selected to obtain from 2001 ASHRAE Fundamentals, Chapter 24, a surface conductance h=6 Btu/(hr-ft^2-°F).

The heat loss through the insulation (conduction) was added to surface losses (convection) to arrive at the total heat flow in each case. For ease in calculation, a one-foot length of pipe was analyzed with 4-inches of insulation, and a total allowable BTU loss was established based on temperature criteria. Following that, elapsed time was calculated for the given temperature drop from 36°F (coldest expected incoming temperature) to 32°F (onset of freezing) within the pipeline, under zero-flow condition on winter design day (-50°F). The results of the analysis showed that for 4″ insulation thickness, 12.6 hours of protection, or 0.32 degrees F per hour was provided.

Because zero-flow conditions are not likely to last longer than 12 hours and the pipeline can be drained in the event of major component/pipeline failures/damage resulting in extended downtime, 4-inches of insulation was the design recommended for the pipe.

Welding Codes. Because of the unusual service and construction requirements in addition to severe environmental limitations, welding quality was judged to be critical to the success of the Pipe Bridges. Code welding requirements were viewed as an important part of quality assurance and Project specifications were developed based upon AWWA (American Water Works Association) and AWS (American Welding Society) requirements with modifications.

- For shop welding, qualification of welders and procedures is addressed in, *AWWA C200, Steel Water Pipe - 6 In. and Larger* that references *ASME BPVC SEC. IX, Qualification Standard for Welding and Brazing Procedures, Welders, Brazers, and Welding and Brazing Operators.*
- For field welding, qualification of welders and procedures is addressed in *AWWA C206-03, Field Welding of Steel Water Pipe* that refers to *AWS D1.1\D1.1M Structural Welding Code-Steel.*

Notch-toughness. To provide a measure of resistance to brittle fracture both base metal and welds were designed for notch toughness; welding procedure specifications (WPS) were qualified for heat input control because of steel thickness in excess of ½” thickness and low service temperatures (less than 40 degrees F) typical of Alaska. The following graph illustrates welding code requirements for limiting heat input based upon procedure qualification in 3G (vertical) position, uphill progression that qualifies the WPS for all-positions. The PQR was qualified at 20 volts, 260 amps and a travel speed of 10 inches per minute resulting in a calculated heat input of 31.2 kilo-Joules/inch that defines the upper heat input limit of the WPS and production welding. For the FCAW process, the AWS D1.1 Code limits WPS voltage to +/- 7% (or 18-22 volts), amperage to +/- 10% (or 230-290 amps) and travel speed to +/- 25% (or 7.5-12.5 inches per minute). Applying these ranges of values and rearranging the heat input formula to solve for travel speed results in the graph below that can provide necessary welder guidance on the WPS. For example, the graph indicates that a welder operating at 20 volts (y-axis) and 250 amps (x-axis), must control travel speed to be 9.6 inches per minute or greater to avoid application of excessive heat to the production weld.

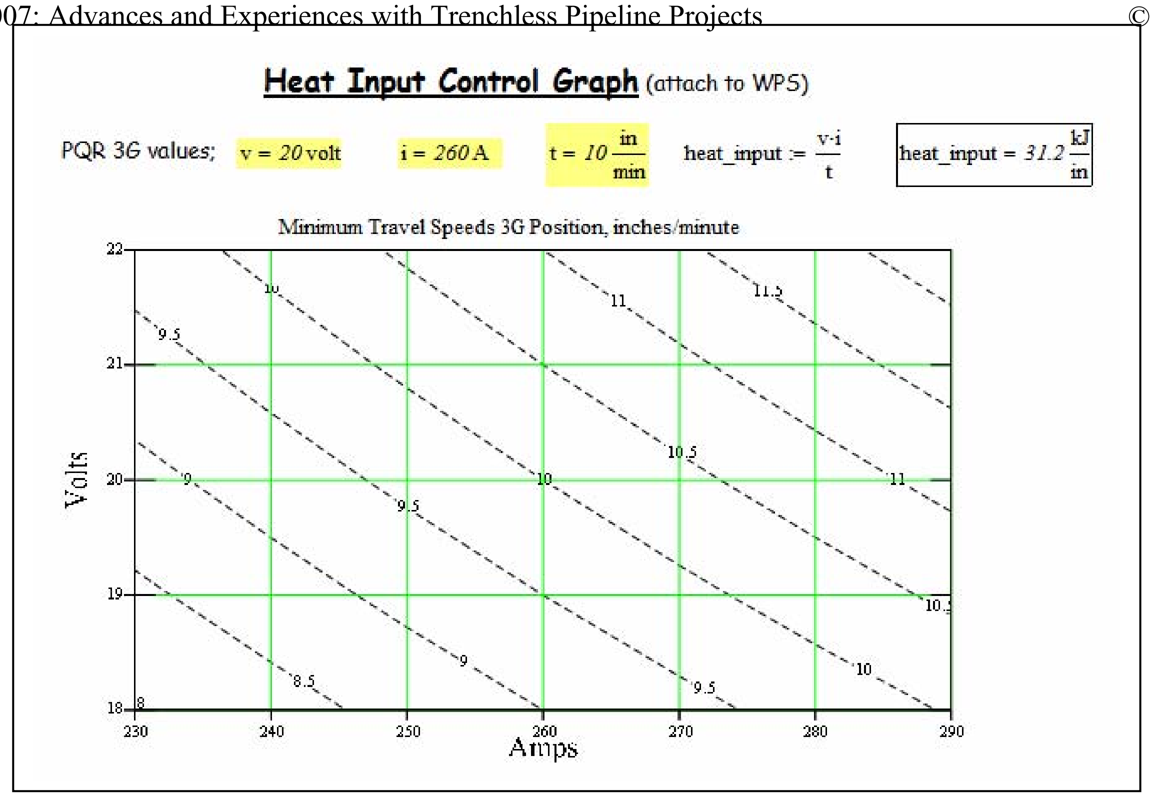

Figure 7 Heat Input Control Graph

Field Welded Joint Details. A high degree of strength and reliability was required for the field welded joints that make up the pipe bridges thus a double welded, complete joint penetration (CJP) butt joint weld was selected.

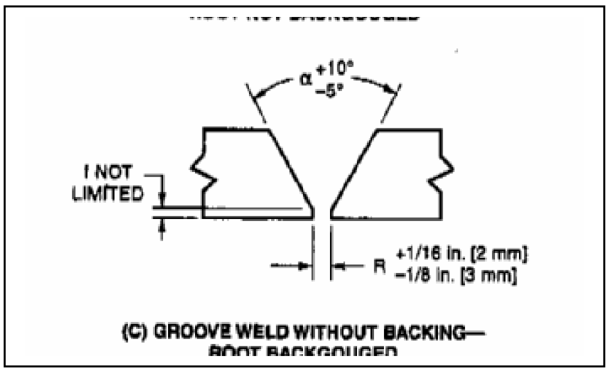

Figure 8 Typical Weld Joint Detail

100% radiographic testing (RT) for X-ray quality field welding was specified in addition to full-time visual inspection by a Certified Welding Inspector (CWI) to acceptance standards for Statically Loaded Non-Tubular Connections in AWS D1.1, Structural Welding Code-Steel. The ASME BPV Code (Section VIII, Division 1, UW-12) permits a joint efficiency, E=1.0 for a double welded CJP butt joint weld that also receives 100% RT; basically this type of joint is judged to be the strongest of any welded joint and has the same strength as the adjacent base metal.

Welding Process. The Contractor selected self-shielded flux cored arc welding (FCAW-SS) process for its portability and high deposition rate in addition to excellent all-position pipeline weld quality capabilities. Filler metal designation E71-T8, and a wire diameter of 5/64-inch was selected for a slag system that favors all-position welding with good, low-temperature impact properties in the weld metal. The electrode desulfurizes the weld metal to a low level, which helps resist cracking and both single and multi-pass application is possible.

Steel Selection. It was recognized that the pipe manufacturing process and the way pipe bridges would be loaded during installation and service required careful consideration of steel selection. High quality pipe can be produced with either the straight seam process, where steel plate is used to form cylinders or by spiral seam process where steel coils are helically formed into cylinders and *AWWA C200, Steel Water Pipe - 6 In. and Larger*, was selected for the pipe bridge specification since both manufacturing methods are addressed in this standard. However, many steels listed in AWWA C200-97, Table 1 (ASTM A570, A607, A907, A935, A36/A36M, A283/A283M, and A572/A572M are structural steels in coil or plate form with yield strengths varying from 30-50 ksi) are outdated. In 2000 the steel industry revised the ASTM Standards for coil products. Coils formerly produced per ASTM A570, A607, A907, A935, and A936 have been replaced by new ASTM A1008/A1008M, A1011/A1011M, and A1018/A1018M. Thus AWWA C200-97, Table 1 for specifying steel coils was not advised since this portion is outdated.

Both straight seam and spiral pipe manufacturing methods require cold forming of the steel to make it into a cylinder; cold forming requires that the steel have good ductility. Thus heavy-thickness steel coils for a spiral welded pipe alternative were specified per ASTM A1018, classification SS (Structural Steel) with the following modifications;

- fully killed steel, fine grain practice,
- Yield: 36 ksi , minimum.
- Tensile: 65 ksi, minimum
- Elongation: 26 percent minimum in a 2-inch gauge length.
- Weld-Ability: Carbon Equivalent (CE) <0.45.
- Toughness: 25 ft-lbs at 30 degrees F as determined by Charpy V-notch testing

Chemistry of the steel coils was also controlled as follows:

- carbon content limited to 0.20 percent or less,
- manganese content limited to 1.35 percent or less and
- fine-grained steel making practice could be obtained by requiring aluminum content of 0.02 percent or greater.
- Phosphorus content limited to .025 percent or less.
- To improve notch toughness, the sulfur content was limited to 0.015 percent.

Ring Girders. The ring girders, that must support an average of 120 kip design loads, were fabricated from steel plates per ASTM A516, Grade 70 with consideration given for limited available space between the pre-stressed concrete bridge girders. The ring girders extend only 4 inches beyond the pipe surface and are built-up from fabricated steel plate webs and rolled steel flanges that are ¾-inches thick.

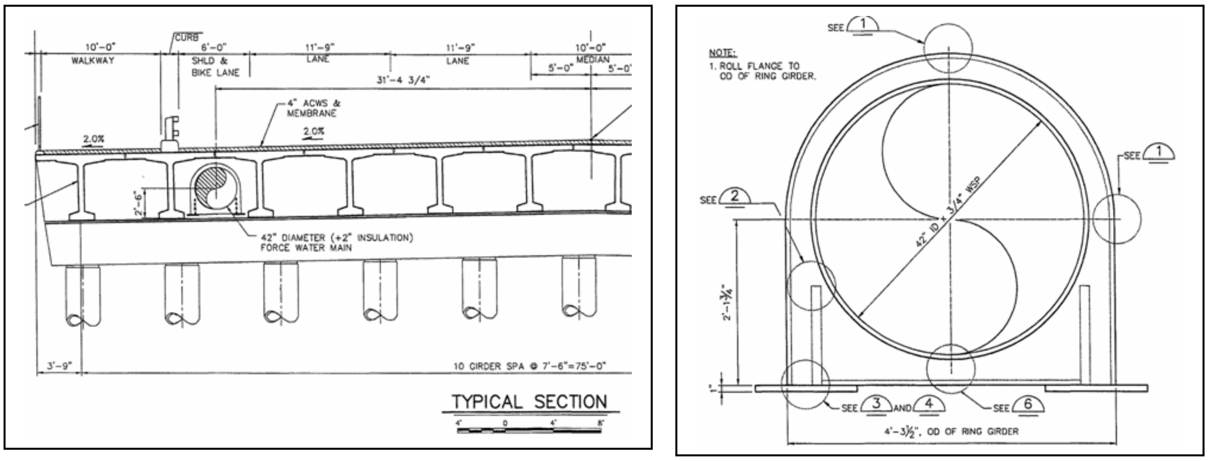

Figure 9 Ring Girder Details

Construction. Construction of the heavy walled pipe presented many challenges ranging from mill fabrication, transportation, and planning.

Road and pipe were at a skew relative to bridge pilings (25 degrees on the North bridge and 20 degrees on the south bridge), so it made fabrication of pipeline components on the bridge complicated because of the geometry. The nozzles for the CARV's, the lower base plates, and the pipe bumper bolt patterns included the skews. The placement of the CARV's on the bridge pipe was planned such that the piping would be centered in the diaphragm above the pile cap. At one location, this did not occur.

The pile caps included a 2% slope away from the center. The slope on the pile cap resulted in some confusion during installation. Some of the anchor bolts for the ring girder lower base plates were not installed level in the vertical position. Instead they were installed perpendicular to the pile cap because the base plate bolt pattern template was laid level with the slope of the pile cap. Leveling nuts were required for the anchor bolts because the lower base plates for the pipe ring girders were required to be level. The leveling nuts were grouted into place once the ring girders were set on the level base plate.

The vertical curve of the pipe matched that of the bridge design. The pipe ring girders were fabricated on the pipe to include the bridge geometry. Centerline elevations for the bridge pipe allowed a clearance of 12-inches between the exterior pipe insulation and the bridge T-beams above. After construction, the clearance proved closer.

The heavy walled bridge pipe was fabricated by a major pipe supplier's steel mill in the lower 48. The approved steel had a yield strength higher than the required specifications (specs called for ASTM A1018, HSLAS gr 50 steel, with a 36 ksi minimum yield; 58 ksi yield steel was used for fabrication). The mill transmission broke down numerous times during fabrication resulting in much rejected pipe. More rigorous testing of the factory fabricated pipe was requested as a result of a field visit to the factory by the engineer. The choice was given to the contractor to either fabricate new pipe, or test the existing pipe to ASME Section VIII paragraph UW-51 instead of UW-52 as specified because of the high reject rate.

All the factory weld records (VT results and RT films) were requested by the engineer. As of this writing, they have not been supplied. However, field RT testing of the butt welds adjacent to the factory spiral weld found spiral weld factory defects, which was subsequently repaired.

Once the pipe was fabricated, it was shipped to Fairbanks, AK where an approved alternative insulation was applied. It consisted of 3-inches of factory applied polyurethane insulation with a factory applied galvanized steel exterior. The physical characteristics of the insulation and jacketing were superior to the specified product, and were much less labor intensive to install. Unfortunately, when the finished product was being driven down to Anchorage, the brakes on one of the trucks caught on fire. This caused fire damage to three spools of the bridge pipe. As a result of this incident, the pipe was inspected for damage, the truck driver interviewed for observations, metallurgical tests were done to the pipe, and lining, coating and insulation was repaired before accepting the pipe.

Once the bridge pipe was onsite, the pipe was laid out and fit-up for welding on the ground at a staging area. There were three bridge pipes welded on the ground from abutment to abutment for the three bridge pipes. One of the complicating issues was fit up of the butt ends of the pipe. AWS D1.1 requires a 1/8-inch max root opening for tolerance on fit-up. However, AWWA C200 allows for a +/- 2-inch tolerance in length. Because of the large allowance on length and minimal weld opening tolerance, fit-up of the pipe was lengthy and labor intensive.

During the design, it was decided to use a polyurethane lining in lieu of a cement mortar lining to reduce the weight of the bridge pipe. As previously noted, the design called for full penetration, open root butt welds. The design specified that the factory applied polyurethane lining be protected from weld spatter. Fire blankets were taped around the interior and exterior of the pipe in the proximity of the joints to protect the lining and coating. However, weld spatter made its way into the lining and caused extensive lining damage that needed to be tested and repaired.

Closing. The sensitive nature of the creeks, combined with costs and lack of local contractor experience, required a unique pipe bridge design that incorporated continuous self supporting, 120-foot long spans between bridge piers and abutments. The insulated, heavy walled steel pipe, along with the bridge and pipe geometry created some problems during fabrication and construction. With proper inspection and specifications, these challenges were overcome and the bridge pipe was installed.

Design and Construction of Large Diameter Steel Yard Piping for a "Fast Track" Design-Build-Operate (DBO) 100 MGD Water Treatment Plant

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Abstract

The San Diego County Water Authority (Water Authority) selected CH2M HILL to design, construct, and operate its new 100 mgd Twin Oaks Valley Water Treatment Plant (TOVWTP). The project includes 21 major structures built around an existing 22 million gallon reservoir on 45 acres of land. By contract, the $157 million dollar project must be designed, constructed, and operational within 30 months. This requires a "fast track" design and construction schedule, multiple pre-purchase contracts with vendors and suppliers, and separate foundation packages for many of the structures. Mass excavation and earthwork, based on the 30-percent design package, began approximately three months after receipt of the Notice to Proceed. The design is complete and construction is moving at a very rapid pace to ensure that the 100-mgd TOVWTP will be operational by April 2008 to meet regional treated water needs for San Diego County.

Project Background

The Water Authority is the primary wholesaler of imported water for San Diego County, distributing water to 24 member agencies that in turn deliver water to individual homes and businesses. More than 90 percent of the total water supply for the three million residents of San Diego comes from imported water supplied by the Water Authority.

In the last four years, the region has experienced peak-demand conditions exceeding the available treatment capacity. These events required that treatment facilities operate above their rated capacity, reducing the reliability of water supply to San Diego. The Water Authority typically implements capital facilities using design-bid-build (DBB) procurement. However, after evaluation of DBB and the design-build-operate (DBO) procurement method, the DBO method was used to address the immediate need for additional treated water capacity in the region. The DBO procurement method was utilized for the TOVWTP for the following reasons:

- The immediate need for the water treatment plant. The Water Authority determined that they could deliver this plant approximately one year sooner compared to a DBB procurement process.
- The ability to assign risk to those most capable of efficiently managing it. The Water Authority specializes in conveyance and does not have a history of water treatment operations. Using DBO, the operations risk is transferred to an entity that has the capacity and background for successful O&M.

- Potential for cost savings using DBO over DBB with contract operations. Preliminary studies performed by the Water Authority indicated potential capital cost savings of between 13 and 33 percent using DBO procurement.

The Water Authority issued a Request for Proposals in December 2004 to design, construct, and operate a 100-mgd water treatment plant. On October 20, 2005 a Notice to Proceed was issued to CH2M HILL, the DBO contractor, based on the Service Contract executed by the Water Authority and CH2M HILL. Many of the requirements identified in the Service Contract directly, or indirectly, affected the design, procurement, and installation of larger diameter pipe. The requirements included the following:

- Fabricate and install steel conveyance piping to and from the water treatment plant in accordance with the Water Authority's standard specification.
- Fabricate and install steel piping within the water treatment plant facilities in accordance with CH2M HILL's specifications.
- Construct connections to Pipelines 3, 4, and 5 in the Water Authority's Second Aqueduct.
- Design, fabricate, and install the entire large diameter yard piping within a 15-month period to accommodate full plant operation by April 15, 2008.

This paper will present challenges faced and lessons learned by the designer, pipe supplier, pipe installation contractor, and general constructor during design and construction of the large diameter yard piping. Significant challenges discussed include: 1) Design Challenges; 2) Procurement and Installation Challenges; and 3) Schedule Constraints.

Design Challenges

For the designer, DBO procurement methods may have additional project coordination challenges associated with multiple specifications (as in the case of this project), multiple construction packages and vendors, changes during construction, and material changes created by suppliers.

Water Authority and DBO Specifications

The Service Contract identified portions of the plant facilities that would be maintained and operated by the Water Authority and facilities that would be maintained and operated by the plant operator. The Water Authority controls flow in and out of the plant. CH2M HILL must produce water within the facility accordingly. This stipulation allowed CH2M HILL to utilize two similar, but different, steel pipe specifications, Water Authority and DBO contractor. CH2M HILL noted that the Water Authority's specification is historically based on long, high pressure transmission installations, as compared to the very low, near atmospheric conditions typical of most water treatment plants. As a result, CH2M HILL designers and constructors determined that fabrication efficiencies could be achieved by using two specifications for large diameter steel pipe while maintaining the overall intent of the Water Authority specifications and a high standard of care.

- Water Authority specifications for steel pipe require in-plant inspection for material verification, pipe welding, weld repairs, tape and mortar lining application, hydrostatic tests, and final review of fabrication and quality control documents prior to shipping.

- The DBO specifications require that the pipe fabricator "certify" that the material, pipe welding, weld repairs, tape and mortar lining application, and hydrostatic test meet specified criteria. The DBO contractor performs a quality assurance in-plant inspection during pipe fabrication and prior to shipment of the pipe.

Two fabrication processes were developed. However, the two fabrication policies or specifications caused fabrication and inspection difficulties for CH2M HILL's in-plant inspectors and Northwest Pipe Company (NPC). Ultimately, the Water Authority inspection procedures, fabrication standards, and specifications were used for the yard piping.

Multiple Design and Construction Packages

The design effort resulted in nearly 800 design drawings and 200 specifications that comprised approximately 40 procurement and construction packages in five months with design work performed in offices throughout the firm and multiple subcontractors providing support. Three foundation packages for foundation grading were finalized two weeks after kicking off the design development phase. Large diameter welded steel pipe, with sizes ranging from 42-inch to 96-inch diameter, was one of the first procure and construction packages for this project. The large diameter steel yard piping designs required coordination with 132 of the 800 design drawings.

Pipe alignments were revised until the 90 percent design submittal. Yet to meet the aggressive construction schedule, priorities were assigned to short sections of yard piping to expedite fabrication and shipment to the site. Many wall spools were fabricated, shipped, and installed early in structural floor slabs and walls.

Early in the steel pipe design phase, criteria for internal pressure, pipe handling, external loading, depth of cover, trench side support, and reinforcement of fittings were established before a complete analysis of all potential conditions could be conducted. Internal pressures were determined to be low, less than 50 psi, even when considering surge pressures, and did not govern wall thickness criteria. Evaluation of pipe handling requirements for 40-foot sections of cement mortar lined steel pipe resulted in a diameter to wall thickness ratio (D/t) of 200. Since the final grading plan was not finalized, the depth of cover was estimated and construction loads were considered for external loading. Under normal conditions, trenches would be cut for the yard piping and the pipe installed with compacted pipe zone material. The constrained site conditions and fast-track construction necessitated alternative pipe zone materials. Controlled low-strength material (CLSM) and both reinforced and unreinforced concrete was the primary pipe zone materials used. The low operating pressures expedited fitting fabrication because no additional reinforcement was required at outlets and tees.

Implementation of Design and Construction

The constrained site required that sections of yard piping be temporally supported until encased in reinforced concrete or CLSM. Wall thicknesses allowed for unsupported spans of approximately 50 feet. However, the yard piping was supported at 20 to 30-foot intervals during construction and prior to placement of pipe zone material as shown in Photos 1 and 2.

Photo 1 – 84-inch Yard Piping

Photo 2 – 72-inch Yard Piping

The DBO contracting method created opportunities for continuous value engineering analysis. Rapid and consistent material escalation required constant vigilance and response to maintain the project budget. Value engineering options are not typically evaluated under a DBB project once it is in construction, but with DBO delivery, CH2M HILL designers and constructors were striving to maintain budget while continuously ensuring overall quality and compliance with the Service Contract. Continuous value engineering efforts resulted in the need to redesign pipe alignments, revise plan and profile drawings, and revise yard piping shop drawings. In one instance wall pipe anchored to a valve vault wall resulted in vault wall design thicknesses of 2 feet-8 inches. Following value engineering analysis, the design was changed to install wall sleeves, provide harnessed mechanical couplings on the 72-inch and 84-inch piping, and reduce the wall thickness by 12 inches. This resulted in a $100,000 construction cost savings.

During the yard piping submittal process, alignments and designs were changed to match the most current design. On multiple occasions, the location of structures changed to facilitate construction access and site grading constraints. When the location of structures changed, plan and profile drawings were revised and modified for the new construction plan, and in many cases these changes resulted in revised shop drawings and, in some instances, changes in the fabrication and inspection schedule.

In an effort to reduce the cost of steel pipe fabrication and meet a critical early delivery schedule, sections of yard piping initially designed to be concrete encased were tape wrapped without a mortar overcoat as shown in Photo 3. This fabrication method was acceptable according to the contract specifications. However, it became apparent that any savings in fabrication cost were canceled by the extra care and handling of the tape wrapped pipe. Eventually, the construction and design team agreed to apply the mortar overcoat, thereby resulting in less installation time for the encased pipe even where concrete encasement was required as shown in Photo 4.

Photo 3 – Tape or Epoxy Coating

Photo 4 – Tape and Mortar Coating

Submerged Membrane Manufacturer Pipe Lining Requirements

After initial design and approval of submittals for the raw water pipelines between the fine screen filters at the untreated water flow control facility and the submerged membrane facility, the membrane supplier informed CH2M HILL that cement mortar lining for the feed piping would not be appropriate for protecting membrane integrity. CH2M HILL evaluated lining alternatives and determined that plant applied epoxy would be the most suitable and cost effective lining. Shop drawings were revised and CH2M HILL and NPC negotiated a change order to substitute plant applied epoxy for cement mortar lining. NPC then subcontracted the plant applied epoxy to a second tier contractor. By subcontracting the lining midway through the fabrication process, approval, fabrication, delivery, and construction challenges resulted. First, CH2M HILL designers and constructors had to select and approve the epoxy lining. Then the CH2M HILL team had to approve the applicator's epoxy application procedures and quality control program. By subcontracting the epoxy lining, CH2M HILL needed assurance from another vendor that the fabrication and pipe delivery would not be compromised. Design of future membrane filtration facilities should include advance verification of lining requirements to maintain membrane system warranties. This example is emblematic of the flexibility required for successful fast-track DBO delivery.

Procurement and Installation Challenges

DBO delivery methods create both procurement and installation challenges as well as opportunities for designers, construction crews, suppliers, and owners. The DBO team must work together to meet critical construction milestones.

Procurement and Installation of Insulation Flange

Plant piping was insulated from the Water Authority transmission system, which consists of two large diameter pipelines, a 90-inch prestressed concrete cylinder pipe (PCCP) and 96-inch mortar coated steel pipe (MCSP) within its Second Aqueduct. The treated water connection isolation was designed with a buried insulating flange assembly that was installed during a 10-day aqueduct shutdown period. Prior to actual installation of the assembly, significant coordination between designers, pipe fabricator, quality control personnel, and construction staff was required because this assembly was an integral component of the Pipeline 4 connection. The insulating flange was fabricated and factory assembled according to Water Authority specifications and details. Following assembly, the flange was tested at the NPC plant, then shipped to the site and tested again following unloading (Photo 5). The test indicated that eight bolts failed. The failed bolts, sleeves, washers, and nuts were removed and replaced. The common observed failure was a result of the sleeve compression against the washers and nuts and bolt threads cutting through the sleeve as shown in Photo 6. The damage to the sleeves appeared to be caused by the assembly method that utilized both a static wrench and an impact wrench. Subsequent sleeve, washer, and bolt installation was performed using two impact wrenches.

Photo 5 - Bolts Retested at Site

Photo 6 - Damaged Sleeves

Photo 7 shows the assembly being set in the 90-inch pipeline. After the assembly was welded in place using two butt straps, each bolt was tested again as shown in Photo 8.

Photo 7 - Insulating Flange Assembly

Photo 8 – Testing Of Assembly

Bedding and Backfill

Early critical installation procedures that required filter fabric and crushed rock pipe embedment (see Photo 9) were replaced with CLSM as the pipe embedment material. The construction site had limited opportunity for trench excavation due to structure construction adjacent to pipe alignments and construction vehicle circulation needs. Installation of the crushed rock with filter fabric proved problematic because limited trench depths required embankment type pipe installation. The flowable properties of the CLSM reduced the pipe zone installation effort resulting in significant schedule improvements. The mix design had a six-inch slump requirement, and the mix contained cement, fly ash, water, aggregate, Daravair 1000 admixture, and air. Aggregate sizes ranged from 100 percent passing a 3/8-inch sieve to two percent retained on a 200 sieve. The material has a 28-day compressive strength between 50 to 150 psi and resembles a sandstone type material when cured. Photo 10 shows CLSM being placed in the pipe zone.

Photo 9 - Granular Embedment

Photo 10 - CLSM Placed In Narrow Trench

Photo 11 shows CLSM placed around the pipe using formwork and Photo 12 shows vertical sides of the hardened CLSM that encapsulates the steel pipe. Structural backfill was compacted in even lifts around the CLSM.

Photo 11

Photo 12

CLSM Placed with Formwork

Photos 13 and 14 show CLSM being placed in the pipe zone and the totally encased yard piping.


Photo 13


Photo 14

CLSM Placed in Trench

Schedule Constraints

Aggressive design and construction schedules typically associated with Design-Build (DB) and DBO procurement create challenges for the design and construction team.

Critical Path Schedule Challenges

Immediately following the Notice to Proceed, the entire design team was mobilized to begin detailed design. Most of the DBO team leadership had participated in the proposal and selection process. This was a key factor in establishing continuity and in providing early direction to the new design and construction team members. Concurrent with the formation of the design team, the on-site staff of construction managers, inspectors, superintendents, staff engineers, document control staff, and project administrators was assembled at the TOVWTP site.

The initial schedule required that construction first begin on the Untreated Water Pump Station (UWPS) and delivery of the associated large diameter wall pipe was required four months following the start of design. The design team had five months to complete the design starting at a concept level set of documents that were developed for the Service Contract. Large diameter pipe designs that included plan and profile drawings with sufficient detail for pricing and pipe submittal preparation were prepared in one month. These drawings were coordinated with a 30-percent complete water treatment plant design.

Large diameter steel pipe was needed immediately following rough grading. Based on a preliminary yard piping plan of the large diameter pipe and corresponding pipe profiles, CH2M HILL contracted with NPC. Immediately following contract negotiations, NPC submitted shop drawings for critical pipe sections. Delivery of the first pipe sections coincided with completion of the rough grading as shown in Figure 1. Fabrication of the large diameter pipe required an aggressive schedule of ten weeks for shop drawing preparation, approval, fabrication, inspection and delivery to maintain critical path development of the UWPS and, subsequently, the Treated Water Flow Control Facility (TWFCF) for the critical Pipeline 4 Tie-in. The Water Authority Aqueduct shutdowns are scheduled two to three years in advance. Missing available shutdown windows is not an option considering that water supply to the entire San Diego County can be interrupted. The first section of 72-inch diameter pipe was shipped to the site 11 weeks after a purchase order was issued to NPC.

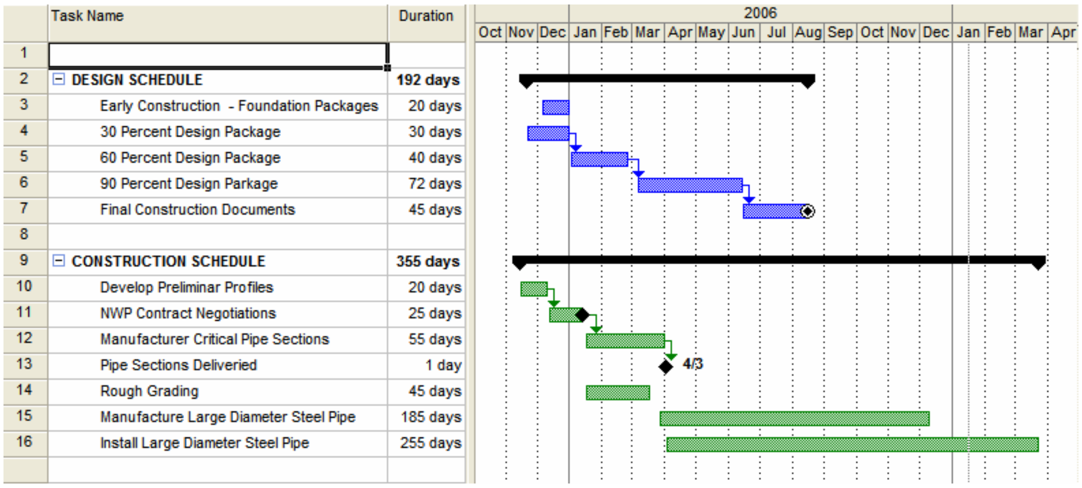

Figure 1 – Early Design and Construction Schedule

Pipe installation challenges associated with fast track DBO contracts

Design, fabrication, installation, and construction of yard piping require early design decisions that can be easily modified during fabrication and installation of the yard piping. Engineers must evaluate the design conditions at the 30-percent level and weigh the cost of waiting for additional criteria that may become available at later design stages. Depth of cover, building locations, utility crossings, and design criteria are all susceptible to changes and must be dealt with by the design engineer following each design submittal and evaluation of estimated construction costs. At each phase, the design criteria is re-evaluated again to determine if construction costs can be reduced without sacrificing quality of construction.

Lessons Learned

DBO procurement methods provide benefits to both owners and DBO contractors. However, there are inherent risks and coordination challenges with DBO procurements. Some of the DBO procurement lessons associated with design, fabrication, and installation of the large diameter steel pipelines include:

- DBO contracting methods can result in multiple design changes to facilitate construction activities and meet schedules while maintaining the construction budget for the project.
- Multiple design changes will occur and piping submittals and fabrication work will be modified to meet the construction schedules.
- Design of future membrane filtration facilities should include advance verification of lining requirements to maintain membrane system warranties.
- Large diameter steel yard piping with low operating pressures should be manufactured based on an acceptable or minimum diameter to wall thickness ratio.
- Although a mortar overcoat adds fabrication time and cost, the addition of the overcoat saves installation time at the site and thereby offsets the cost.

- Full time in-plant inspection, if required by contract, should be implemented at the beginning of pipe fabrication.
- Wall pipe sections should be short sections with lengths less than 10 feet.
- Box outs in walls should be considered so wall pipe can be installed later.
- Piping plans should include butt strap connections rather than lap welded joints to allow for changes in location of structures.
- Controlled low-strength material should be used to expedite pipe installation.
- Design-build and design-build-operate contractors should work closely with local engineers familiar with the Owner's specifications to fully understand the implications of Owner furnished specifications on design-build construction costs.

OVER DEFLECTION OF 48-INCH STEEL WATER LINE

Gregory J. Henry, P.E.[1]
Aaron N. Drucker[2]

The contractor installing a 48-inch steel water line filed for bankruptcy and vacated the project after installing approximately 90% of the pipe, but prior to any testing. The contractor's surety company hired another contractor to complete the project, which consisted of approximately 4,800 linear feet of tape coated and mortar lined steel pipe along the busy roadways of Almeda and Holcombe near Houston's Texas Medical Center.

Upon arriving at the site, the completion contractor notified the City of Houston of potential problems with the pipe already installed, particularly, excessive deflection or "egging" along the vertical axis. Following this discovery, the City contracted with Lockwood, Andrews & Newnam, Inc., (LAN) and invited the pipe manufacturer to perform an internal inspection to assess the situation and recommend a repair solution.

The inspection revealed approximately 141 of 235 pipe sections installed (approximately 3,035 LF, or 60% of the total pipe installed) had deflected beyond the 3% allowed by Specifications. The inspection also revealed several segments with dented cylinders, one with a puncture through the cylinder, and seven with severe longitudinal cracks which are indicative of damage to the cylinder. Pipe sections were observed to have deflected up to a vertical diameter of 44-inches, approximately 8.3%.

City of Houston Specifications state that the maximum allowable deflection for mortar-lined steel pipe with a flexible coating is 3% of pipe diameter. This is consistent with AWWA M-11 requirements. (In this case: 3% of 48-inch = 1.44-inch maximum allowable deflection). There are two major reasons for limiting the amount of deflection allowed. First, the pipe relies on a circular shape to support the external loads. Second, excessive over deflection can cause cracking of the interior mortar lining which could lead to corrosion.

However, because much of the project limits were within the roadway, and had already been repaved, correcting the pipe deficiencies would have a major impact to Texas Medical Center traffic. LAN and the City agreed on a variance in the maximum deflection, and accepted pipe with up to 4% deflection. A total of 58 pipe segments exceed this deflection. It was required that any pipe to be re-rounded would still meet the original specifications of 3% maximum deflection.

The observed deficiencies also led to other concerns regarding possible pipe coating damage and the potential for long-term pipe and pavement settlement.

This paper presents the results of internal and external pipe inspections, pipe embedment and backfill testing and discusses the likely causes of over deflection in this flexible pipe. The paper also describes the repair methods used, repair cost, and lessons learned in the process.

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I. Project Background

The 48/30-inch water line along Almeda and Holcombe, file number S-0900-90-3, consists of approximately 4,800 linear feet of 48-inch (inside diameter) and 1,323 linear feet of 30-inch steel water line along Almeda and Holcombe from north of Brays Bayou to Cambridge Street.

The construction began in July 2004. The contractor completed approximately 85% of the water line installation, however, defaulted on their contract after filing for bankruptcy prior to final completion and testing. The contractor's bonding company then contracted with Texas Sterling Construction, Inc. (TSC) to complete the project.

The water line is constructed from 0.235-inch thick steel pipe with tape coating and cement mortar lining (CML), manufactured by Northwest Pipe Company (NW Pipe). The tape coating is applied by the manufacturer in a three layer system consisting of a primer, an inner layer of corrosion preventive tape, and an outer layer for mechanical protection. The mortar lining is comprised of Portland cement, sand, and water, which is centrifugally compacted by the manufacturer to a thickness of approximately ½," with a manufacturer's allowable tolerance of -1/16" to +1/8". A cut-through schematic of the pipe construction is included as Exhibit 1.

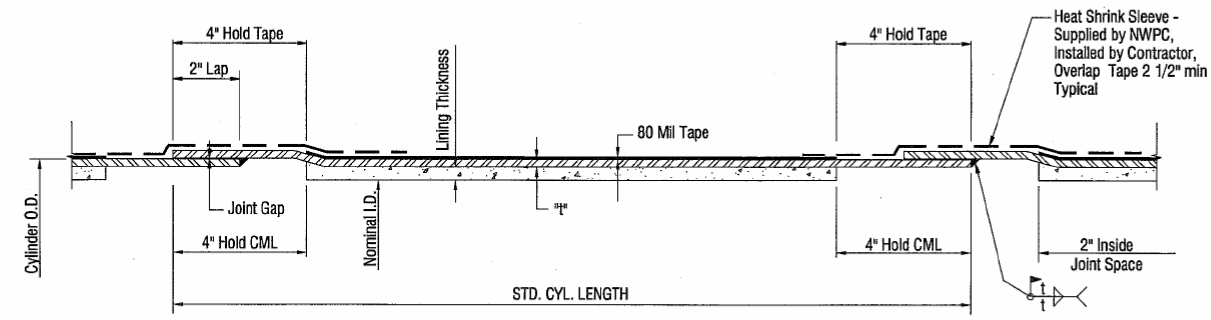

Exhibit 1: Cut-through schematic of a typical tape-coated and cement mortar-lined (CML) steel pipe section.

City of Houston specifications for large diameter steel water mains state that the maximum allowable deflection for mortar-lined pipe with a flexible coating is 3% of pipe diameter, which is consistent with AWWA M11 requirements. (In this case: 3% of 48-inch = 1.44-inch maximum deflection). The 48-inch measurement is for the flow area of the pipe, meaning it is an inside diameter measurement taken from mortar lining to mortar lining.

There are two major reasons for the deflection limit. First, the pipe relies on a circular shape to support the external loads. Second, excessive over deflection can cause cracking of the interior mortar lining which could lead to corrosion.

While completing the project, the takeover contractor noticed the 48-inch pipe to be deflecting, or "egging," several inches along the vertical axis. Following this discovery, the City contracted with LAN and invited NW Pipe to perform a preliminary internal inspection to verify pipe measurements. No over-deflection was noted in the 30-inch pipe sections, and they were not available for inspection by LAN.

II. Inspection and Recommendations

On 2/24/06, the City, LAN, NW Pipe, the Surety and their completion contractor met to discuss the situation.

LAN and Northwest Pipe Company inspectors conducted field investigations of the pipe by measuring the vertical and horizontal diameter of each pipe section. Measurements were taken using a tape measure across the vertical axis. This inspection revealed approximately 141 of 235 pipe sections (60% of the total number of pipe sections for the entire project, approximately 3,035 LF) which had deflected beyond the 3% allowed by project Specifications. The inspection also revealed one segment with a dented cylinder, one with a puncture through the cylinder, and seven with severe longitudinal cracks which may be indicative of damage to the cylinder.

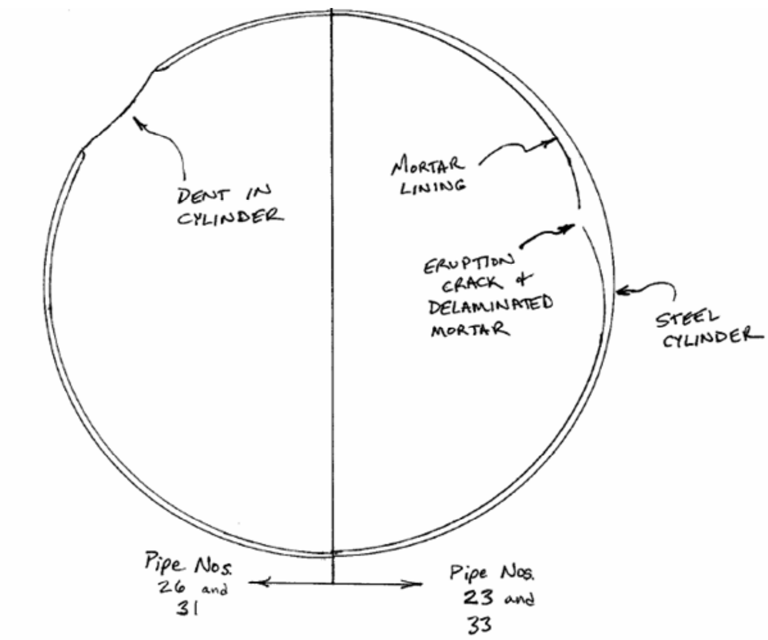

Exhibit 2: A sketch of the type of damage, other than deflection, observed on May 4, 2006 during an internal inspection.

In addition, the inspector noted several instances of substandard joint grouting and welding. In some instances, the pipe was deflected to a vertical diameter of nearly 44-inches, approximately 8.3%.

Due to the flexibility of steel pipe, the cylinder could theoretically be deflected up to 10% without suffering permanent damage (other than cracked mortar lining). However, the manufacturer did recommend that the pipe be re-rounded to reduce the potential for collapse, and cracks in the mortar lining be repaired.

LAN recommended a variance of the allowable maximum deflection to allow up to at 4% deflection as a means to negotiate with the bonding company and to reduce the impact to traffic during repairs. In addition, to account for slight discrepancies in the measurements, due to the accuracy of measuring within the confines of the pipe, and due to slight variations in the thickness of the mortar lining, LAN, NW Pipe and the City agreed to allow an additional ¼” in the measurements to account for these differences.

Based on the variance to the original project Specifications, the Surety, using TSC, would only be required to re-round pipe sections with a deflection of 4%+1/4-inch, or a measurement of 46-inches. A total of 58 segments exceed this deflection. LAN recommended that any pipe to be re-rounded still be held to the original specifications of 3% maximum deflection.

Five additional sections were within the deflection variance, however were recommended to be replaced or repaired due to dents or severe mortar cracking. TSC exposed the steel cylinder in these sections by chipping away the mortar lining for closer examination. Both LAN and NW Pipe agreed that minor dents could be cut out and have an overlapping patch welded to the outside of the pipe (welding both inside and out, similar to a butt-strap). Inspection of the steel cylinder along the pipes with severe longitudinal cracking indicated that no permanent deformation or creases existed in the steel. The contractor was allowed to simply repair the mortar linings in these instances.

III. Possible Cause of over-deflection

The most likely cause of over deflection is lack of support from backfill in the pipe haunches.

This may have been the result of poor compactive effort by the original contractor. It may also be the result of properly compacted material washing out into the multiple excavations left exposed and not adequately protected by the contractor throughout construction and after their bankrupcy.

The City did perform random backfill compaction tests during construction, and while an acceptable repair strategy was being devised, the poject’s Materials Testing Lab was asked to confirm the backfill compaction within the trench zone.

A review of density tests performed during the pipe installation showed no failing tests that had not been corrected. However, according to available soil density reports, the testing lab only performed 2 tests around the pipe or in the bedding zone. According to the City of Houston backfill specifications, successful compaction is to be measured by one test per 40 linear feet along pipe for compacted embedment, and on each lift during backfill. During the repair effort by Texas Sterling, the materials testing lab also returned to test the in situ backfill previously installed as deflected pipe was excavated.

As another attempt to verify the in-situ compaction, the lab was instructed to take soil borings within the trench zone at several over-deflected pipe sections. Due to the time required for contract negotiations and budgetary constraints, only three boring logs were received. The borings as well as excavations performed indicated clay around some of the over deflected pipe sections, which is not consistent with backfill requirements.

IV. Repairs

TSC submited a plan to hydraulically jack the pipe into the designed circular shape after uncovering the deflected pipe and one pipe section on each side down to the haunches, and placing three hydraulic jacks in a pipe segment, one at each end, and one in the middle. This method has been used successfully before by TSC and was also the method recommended by NW Pipe. A mat consisting of a steel plate with either lumber or rubber cushion would be placed in between the jacks and the mortar lining of the pipe to prevent damage to the pipe, as seen in photograph 1. While some of the sections suffered minor damage to the mortar lining due to the jacking, most remained unharmed, and the damaged sections were repaired with grout. The jacks are then manually operated in unison until the pipe is rounded. Once the pipe was re-rounded, 48-inch support stulls were installed at the jack locations, and new sand backfill was compacted in the haunches and up to one foot above the top of pipe in accordance with City Specifications. Depending upon moisture content, the native soil was allowed to be placed above the pipe.

Photo 1: Hydraulic jacks with rubber and steel padding inside MK #35 along Almeda.

After the rounding of the pipe is complete, the trench is backfilled. Every foot the soil is compacted with hand-held rammers and walk-behind plate compactors, as seen in photo 2. Wooden stulls are in place before, during, and after backfilling is complete.

Photo 2: Worker using pneumatically powered hand-held rammer to compact soil around MK #143 as the pipe is backfilled.

On Friday, May 12, at the request of the surety, Texas Sterling brought an excavator with an attachment to assist with the compaction effort. The machine was used in conjunction with the internal hydraulic jacks and walk-behind plate compactors to re-round the pipe and compact the soil. The horseshoe shaped vibratory attachment is approximately 60-inches wide and designed to fit around the pipe and compact soil on both sides of the pipe simultaneously, as seen in photo 3.

Photo 3: Excavator with vibratory attachment, compacting soil around MK #97.

A total of six sections that were beyond repair were required to be replaced. Two contained longitudinal cracks in the mortar lining causing a majority of the lining to delaminate from the steel cylinder. This condition warranted the replacement of the section. The new segments were installed using a butt-strap closure, as seen in photo 4. Four adjacent sections were replaced bell-to-spigot with heat-shrink tape placed over the joints, as seen in photo 5, and did not require the use of a butt-strap.

Photo 4: New segment used to replace MK #23, being installed with the use of a butt-strap between MK #23 and MK #24.

Photo 5: Heat-shrink tape placed at the joint between MK #94 and MK #95.

Two more sections contained dents in the steel requiring either replacement or repair. On 10-inch diameter, circular, holes were cut from the steel cylinder, removing the portion of the pipe which was dented. A 36-inch by 36-inch square patch was cut from a removed segment of pipe and welded over the hole both on the exterior and interior of the pipe, as seen in photo 6. Field-applied tape was placed over the exterior of the steel patch after it was welded into place.

Photo 6: Steel patch being welded over removed portion of steel on MK #31

Several of the segments contained either longitudinal or circumferential cracks in the mortar lining. While some were replaced, most were minor enough to only warrant localized mortar replacement. The largest grout repair was made on Mark #33 where a large portion of the mortar lining had become disbonded from the steel, as seen in photo 7.

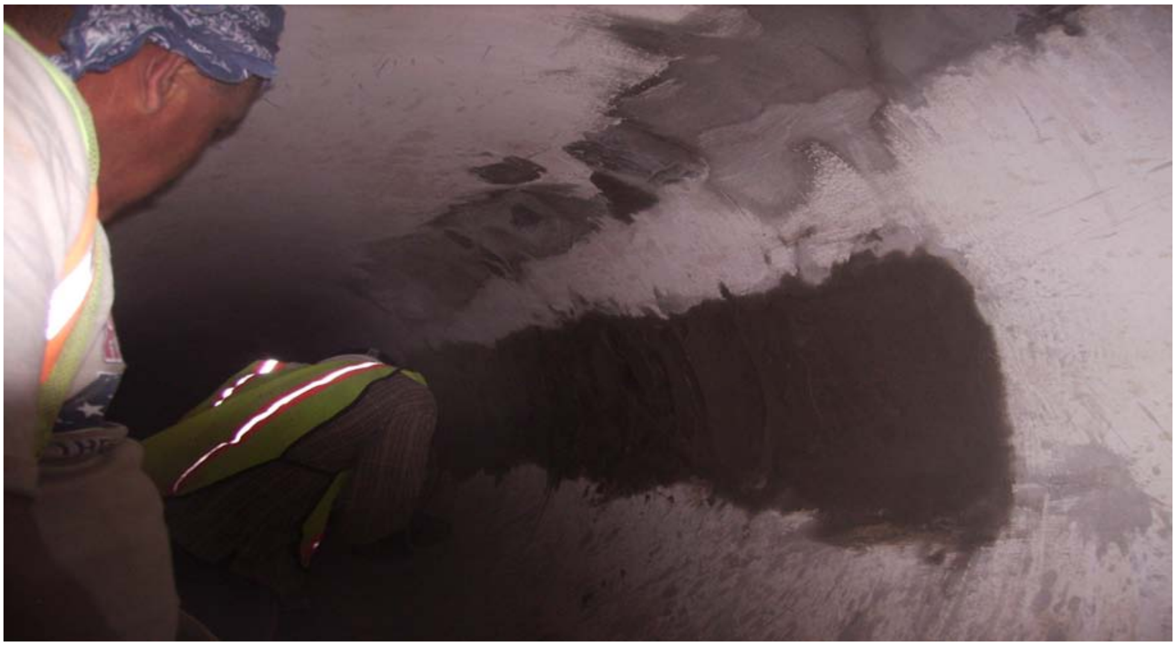

Photo 7: Grout being used to repair a portion of mortar lining which had delaminated from the steel cylinder in MK #33.

On Thursday, May 25, the crew which originally started their work on Almeda, completed their pipe repairs and began removing stulls and cleaning the interior of the pipe. On Tuesday, May 30, both crews finished their re-rounding repairs and replacement. One crew was assigned to another Texas Sterling project, while the other continued removing stulls and cleaning the pipe in preparation for inspection and hydrostatic testing.

In all, 68 complete sections were re-rounded, 1/3 of the length of 4 sections was re-rounded, and six sections were completely replaced. The number of sections re-rounded exceeds the recommended number because while some segments were within the allowable deflection limits, they were in between or adjacent to sections with excessive deflection.

It should also be noted that the LAN inspector on site visually inspected the exterior of each pipe section uncovered for damage to the tape coatings. While the majority of the pipe which was uncovered suffered no tears or other defects in the exterior tape, there were approximately four segments showing pre-existing damage and requiring repair.

V. Post-Repair Inspection

Following the repairs, two LAN representatives entered the line on Wednesday, June 14 to perform a final internal inspection of the line. The objective of the inspection was to check the pipe deflection, inspect grout repairs, and to ensure that the contractor had sufficiently cleaned the line prior to filling it with water for disinfection and testing.

During inspection, segments were randomly selected to have their diameter measured. All sections measured were within the allowable deflection limits, while most measured a true 48-inches. By visual inspection and sounding, all joints and crack repairs appeared to be soundly grouted. The contractor had removed all stulls and cleaned dirt and debris from the line. It was the opinion of LAN that the line was prepared to be filled for disinfection and testing.

VI. Hydrostatic Testing

On Tuesday, June 20, 2006, TSC performed a hydrostatic test of the line, which includes work installed previously by the original contractor which had not yet been tested for leaks. City of Houston Standard Specifications allows 3.19 gallons per inch nominal diameter per mile per 24 hours for O-ring gasketed pipe tested at 150 psi. In this case: 3.19 x 48 x 4800/5280 = 139 gallons allowable leakage.

The line was filled with water and pressurized to 150 psi. This pressure was maintained for an 8 hour period with no measured leakage.

VII. Conclusion

Texas Sterling was able to successfully re-round the pipe into the allowable deflection range, and perform all necessary repairs.

Perhaps the overriding lesson to be learned by all is that the structural integrity of the pipe being used for water lines is paramount in any project. Several parties have a responsibility in the delivering of a functioning water line with undamaged pipe, and it is important that all parties carry out those responsibilities while strictly adhering to the owner's specifications, industry standards, and professional practice. The manufacturer has a responsibility to produce pipe which meets all applicable specifications, the contractor has a duty to properly install and deliver undamaged pipe, and the construction manager and an inspector trained in large diameter water main construction is obligated to oversee all operations of the project and ensure a quality product is being delivered to the owner.

Design and Construction of Denver Water's Recycled Water Distribution System

Matt S. Turney, P.E.[1]

Abstract

In 2001 Denver Water began its efforts to install a Recycled Water Distribution System to bring water from a newly constructed recycled water treatment plant to parks, golf courses, and other large users in Denver. The source water is taken from the effluent of the major wastewater treatment plant serving metro Denver. The water is then treated to bring the water's quality to levels suitable for irrigation and other non-potable uses. Phase 1 of the Recycled Water Distribution System became operational in 2004.

In 2005, Denver Water began the design of Phase 2 of the Recycled Water Distribution System. The purpose of this expansion is to bring water to irrigatable portions of two large areas undergoing redevelopment; the former Stapleton Airport and the former Lowry Air Force Base. In addition to distribution piping, Phase 2 consists of a treated water reservoir and a pump station. Phase 2 is scheduled to become operational in the spring of 2008.

The distribution piping installed during Phase 1 and Phase 2 consists of over 80,000 feet of large diameter pipelines (24" – 54") and over 11,000 feet of small diameter mains (6"-8"). The majority of the pipelines are being installed through relatively older and dense residential areas of the city. The pipelines cut across 3 golf courses and 6 park areas. There are 4 highway crossings and 4 railroad crossings. Relocation of hundreds of utilities, including gas mains, water mains, fiberoptic conduits, and sewers had to be performed ahead of the pipeline installation. This required significant coordination with many different private and public entities as well as an extensive public relations effort.

A total of 12 contracts, totaling approximately $40 million, were used to construct the distribution system. The project was pieced out to facilitate limited design resources and a tight construction schedule.

A number of special features are being added to the pipeline to help identify this pipeline as a recycled water pipeline. These include purple exterior, pentagon shaped valve nuts, and reversal of direction of operation of valves.

Additionally, the pipe was not permitted to be lined with cement mortar due to concerns that it would cause the pH levels in the water to rise. The concern was that

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lime in the cement mortar would leach from the lining during the relatively slow velocities the pipeline would experience in the winter.

Horizontal Directional Drilling was used to install portions of the smaller diameter PVC mains. This method of installation was allowed to compete against traditional open trench installation. This allowed Contractors to weigh the decreased permit and restoration costs versus the setup associated with directional drilling. This method also significantly reduced the level of inconvenience to residents and customers.

The purpose of this paper is to describe the items specific to a recycled water pipeline as well address the coordination and planning required for a pipeline project of this scope.

Background

With water supplies becoming scarcer, non-potable reuse of water has become a viable option for Denver Water to meet a growing demand. Reuse of water is being used throughout the country, particularly in California, Florida, and Arizona. Denver Water began its use of reuse water to satisfy the requirements of the Blue River Decree. This decree allowed Denver Water to take water from the western slope of the continental divide but also required Denver Water to maximize its use of this water. As construction of traditional water storage projects becomes more difficult and expensive, reuse of water has become a more realistic solution.

In the spring of 2004 Phase 1 of the Recycled Water Distribution System became operational. Phase 1 consisted of a treatment plant treating effluent from Denver's primary wastewater plant, an onsite pump station and treated water reservoir, as well as 13 miles of pipelines 6" - 42" in diameter to serve water to industrial users and those with large irrigatiable areas. The treatment plant at a capacity of 30 MGD (expandable to 45 MGD) became the largest recycled water treatment plant in the state of Colorado.

Phase 2 is scheduled to be become operational in the spring of 2008. Phase 2 adds a 30 MGD pump station as well as a 6 MG treated water storage reservoir. An additional 6.5 miles of pipe (54" and smaller) will bring recycled water to two large areas undergoing redevelopment, the old Stapleton airport and Lowry Air Force Base.

Phase 3, scheduled for completion in 2011, will bring on additional customers towards the northeast part of the city including the Rocky Mountain Arsenal (wildlife refuge) and even perhaps DIA (Denver International Airport) and surrounding area. Upon completion of Phase 3 it is expected that the project will supply approximately 17,500 acre-feet of water at a cost of approximately $174 million. This quantity of recycled water will free up enough potable water to serve approximately 35,000 households per year.

Finding Alignments

Unlike utilities that are typically built as development occurs, the recycled water pipelines had to be constructed amongst existing infrastructure. These pipelines are being built in densely populated areas of Denver developed 60-100 years ago. This brings along with it many challenges including heavy traffic, public relations, tight working space and significant interference. As an example I will describe the efforts in determining the alignment for Conduit No. 307, a 54" diameter pipeline linking the Phase 2 pump station and the Phase 2 treated water reservoir.

The area between the proposed reservoir and the proposed pump station is primarily made up of grid-like residential blocks (see Figure 1). This allowed for literally hundreds of potential alignments to navigate this approximately 4 mile stretch. Narrow residential streets (typically 30-36 feet curb to curb) in the area were the obvious choice in an effort to stay away from higher trafficked areas. The alignment selection was made with limited field survey due to the large cost and effort of performing a detailed survey for all the potential alignments. Instead, maps provided by the various utilities were relied upon to give a general sense as to which alignments might be better than others.

Two major features existed in the area that further narrowed the list of favorable alignments. The biggest of these features is an approximately 100 acre area made of 3 Denver Hospitals. These Hospitals are located basically directly between the proposed reservoir and pump station. These hospitals essentially form an area off limits to pipeline construction due to the sensitive nature of the existing utilities and the need for access of emergency vehicles.

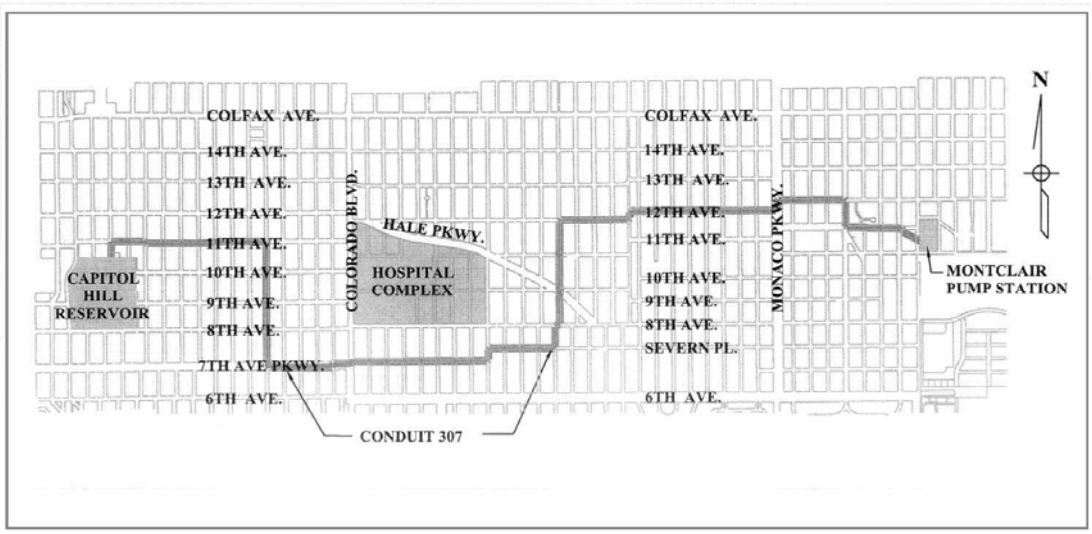

Figure 1 - Map showing grid-like blocks and Hospital Complex

The second of these features was the crossing of one of Denver's busiest Streets, Colorado Boulevard. Given the traffic on Colorado Blvd (over 30,000 vehicles per day) it was a foregone conclusion that we would have to tunnel beneath the roadway. The major obstacle was limited locations for tunnel launching and receiving pits. First of all, most of the streets crossing Colorado Blvd in this area were major streets themselves (Colfax, 14th, 13th, 8th, and 6th Avenues all exceed 15,000 vehicles per day) and secondly, the lesser traveled streets did not line up well from one side of Colorado Blvd to the other. Severe impacts to traffic would have been encountered if the tunneling pits were placed in one of these major thoroughfares crossing Colorado Blvd. Most of the remaining streets were not wide enough to handle a tunneling pit at the skew that would have been required to cross Colorado Blvd where the streets were offset. Upon considering all of the above factors we selected two crossing locations, 12th Avenue and 7th Avenue, for which we would perform a detailed survey.

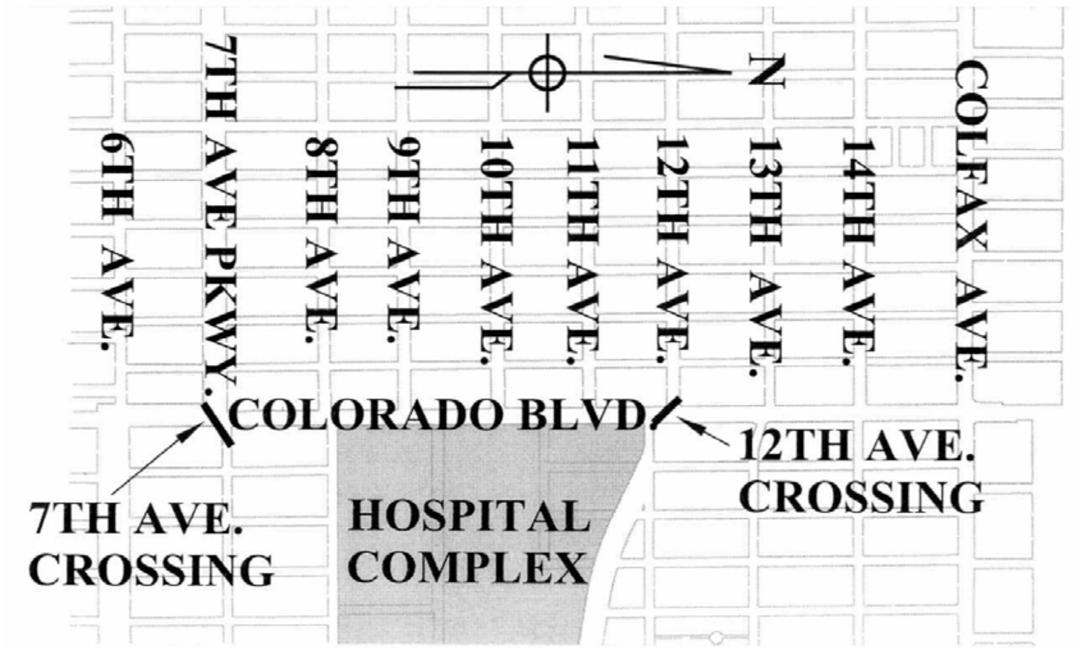

Figure 2 – Plan View of Colorado Boulevard showing potential crossing at 7th Avenue or 12th Avenue

The detailed survey of the 12th Avenue area showed a 90" diameter storm sewer to be more problematic than we originally thought. The storm sewer, made of bricks three courses thick, was installed during the 1930's. We were cautious about crossing this sewer due to prior experience with this storm sewer further downstream on another project. The elevation of the sewer was just high enough that we did not feel comfortable crossing above because of the small cover over Conduit No. 307 that would have resulted. The alignment of the storm sewer was such that a tunnel crossing beneath would have had to cross the sewer twice. In addition, although an alignment along this path would not have lead straight into the hospital complex, it would've ran along its northern boundary. For these reasons, combined with the fact that this crossing was not as perpendicular as we would've like, caused us to look elsewhere.

Closer investigation of the 7th Avenue option yielded what looked like surprisingly few difficulties. The major issue was that to get to 7th Avenue another major street, 8th Avenue, would have to be crossed twice. Additionally, crossing at 7th Avenue would increase the total footage from an idealized 16,000 feet (fairly straight alignment) to 21,500 feet and add approximately $2.5 million to the project total. We also had to consider 7th Avenue's registration as a Historic Parkway, as well as protection of the 100 year old elm trees lining the street. Despite these issues, 7th Avenue was chosen as the best location to cross Colorado Blvd. A soils investigation and utility potholing were performed to obtain the additional information necessary to complete the design.

In order to cross at 7th Avenue, a 48" diameter storm sewer running down the middle of Colorado Blvd would have to be crossed (see Figure 3). Not a big deal itself, but the soils investigation revealed the ground water table along with a split face condition to exist just below the sewer. Microtunneling was not seen as a practical option due to the cost required to bring in the equipment for such a short crossing (140'). Unfortunately placing the tunnel (72" diameter) over the pipeline would have resulted in a cover less than we were comfortable with (approx 4 1/2 feet) for tunneling beneath a major roadway. Because both of these options carried too high of a potential for a mishap resulting in disruption to the traffic above (or even worse), it was decided to approach the City ROW department to discuss the possibility of open cutting Colorado Blvd.

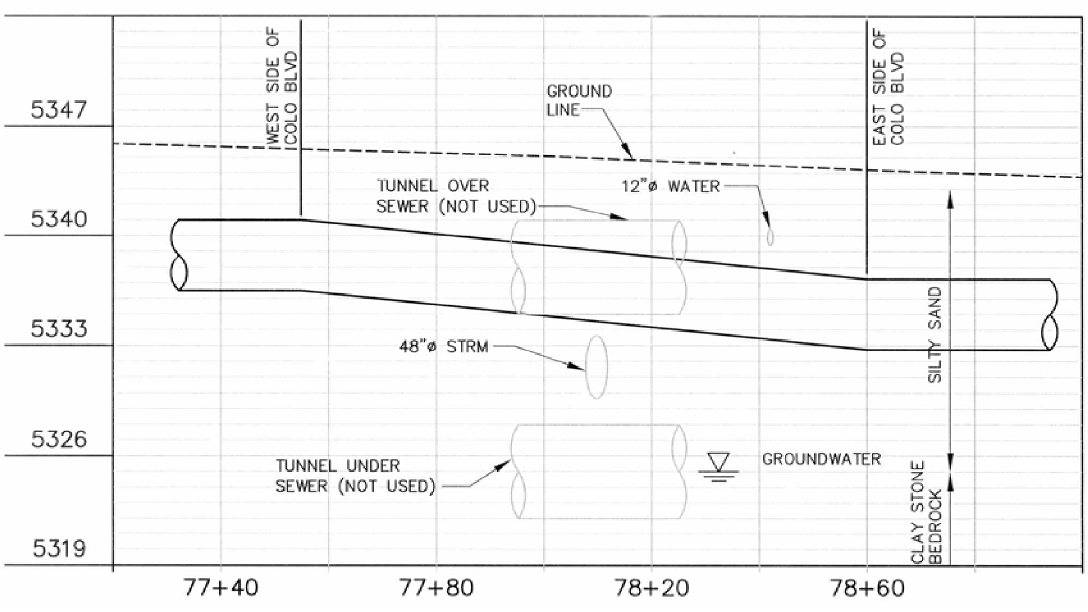

Figure 3 – Pipeline profile on 7th Avenue at Colorado Blvd

Denver Water and the City of Denver reached an agreement allowing an open cut of Colorado Blvd to occur during 5 consecutive nights. The roadway could be closed to traffic beginning at 10 pm and the surface would have to be paved and open to traffic by 6am each morning. This obviously left a very limited amount of time for pipe laying each night. In addition a sound study was required to be performed to establish the pre-existing noise levels and to monitor sound levels during construction. Hotel vouchers were offered to residents whose houses were in areas where sound levels were expected to exceed the legal limit.

Sanitary Sewers

Another feature affecting the alignment choice was the location of sanitary sewers. State regulation governing the use of recycled water requires that recycled water pipelines be aligned a minimum of 10 feet from sanitary sewers and be below the sanitary sewers in elevation. However, it is not the sanitary sewers themselves that present the biggest hassle; it is the sanitary sewer services to each home along the way. The sanitary sewer services on the west half of the pipeline feed into sewers that were built in narrow alleys between the streets. In this case, the sewer services would not have to be crossed. On the east half of the pipeline, the sanitary sewers were installed within the streets themselves. This means that the new 54” waterline would have to go under or over these services. These sewers most often existed just shallow enough (approx 8 ft deep) to prevent the pipeline from being laid above the services, thereby requiring deep and costly installation beneath the services. Generally there were approximately 12 such services (one side of the street) per block. The services could be relocated vertically but only if the proposed elevation of the 54” pipeline allowed such (see Figure 5).

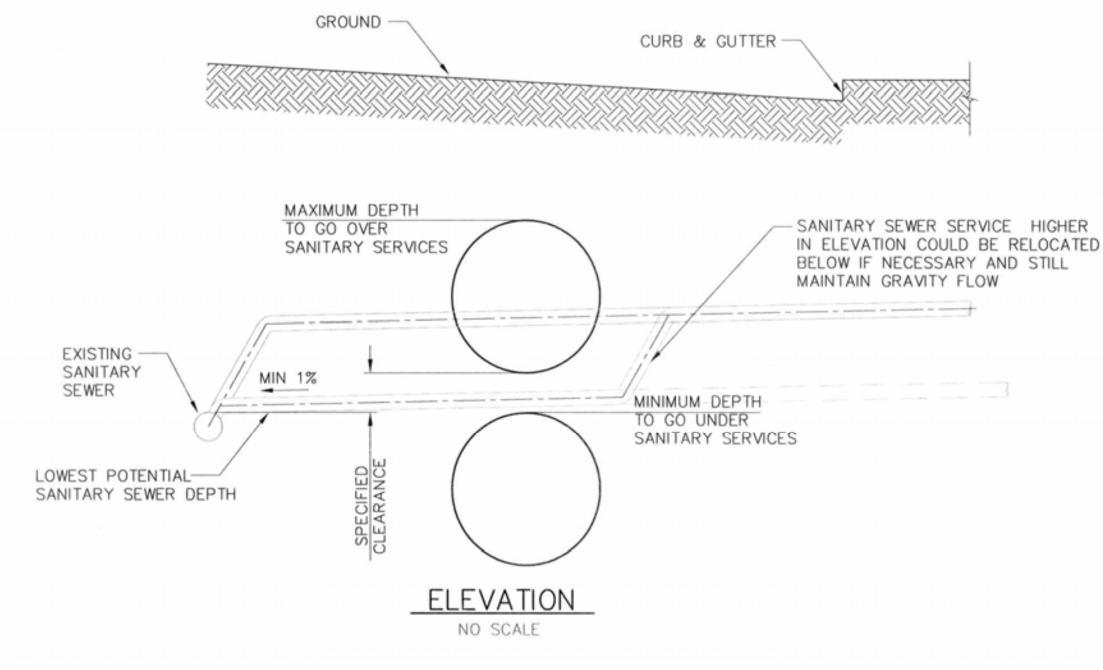

Figure 4 - Sanitary sewer service interference

Since the homes in the older parts of Denver are generally aligned such that their street access (direction toward utilities) is either to the east or west, this problem with the sewer services is only an issue when constructing a pipeline in the north/south direction. The potential conflict with these services was made greater by the fact that the pipeline had to travel so far to the south (and thus back to the north) to cross Colorado Blvd. In the case of Conduit No. 307, this issue could be avoid all together if the movement to get back to the correct northing on the east side of Colorado Blvd was made west of where the sanitary sewers began appearing in the streets rather than the alleys. This is one of the reasons Forest Street was chosen for the north/south alignment.

Following the alignment selection, utilities were potholed, a profile was created, the design was completed and the project was put out to bid. The construction of Conduit No. 307 was awarded to Arapahoe Utilities and Infrastructure of Englewood, CO in February of 2006 for a price of $12.5 million. Northwest Pipe Company of Denver, Colorado manufactured steel pipe for the project. In May of 2006, construction began on the project's three 72" diameter tunnels. Pipe laying began a month later in June. Between 2 and 4 crews worked for the next year to complete the project in May of 2007.

Allowable Linings

It was decided that calcium silicate based cement mortar would not be an acceptable lining material for use in for the Recycled Water Distribution System. This decision was made because of Denver Water's experience that water's pH tends to rise in cement mortar lined pipelines, especially in dead end piping or where flow is low or intermittent. Specifically, this has been a problem in the relatively long water mains that branch from the bulk of Denver Water's system to serve Denver International Airport, DIA. The transmission mains delivering water to DIA are oversized in anticipation for a larger future demand. For the time being, these pipelines are unnecessarily large creating low flows and long travel time. In the case of the recycled water mains, they are sized for the high usage in the summer, resulting in relatively low velocities in the winter.

The leaching of lime from the cement mortar may cause a calcium loss, structural deterioration of the lining, and an increase in pH, calcium, and alkalinity of the water. Besides an unpleasant taste, water with a high pH (above 9.0) can cause skin irritation, both epidermal and internal (esophagus). High pH water also may have a negative effect if applied to vegetation. The pH of irrigation water can adversely affect the availability of nutrient elements. Deficiencies or excesses of certain elements cause stress. For example, consistently high pH causes an iron and/or manganese deficiency in foliage, resulting in yellowing (chlorosis), which ultimately causes leaves and twigs to die.

A rise in pH is more likely to occur in water with low alkalinity. Since rainwater and snow are generally less alkaline than groundwater, this problem is more likely to occur in water distribution systems that are fed from raw water storage reservoirs that receive its' supply from surface runoff such as Denver's.

Horizontal Directional Drilling

Horizontal Directional Drilling, HDD, was allowed to compete against the traditional open cut method for installation of some of the smaller diameter recycled water distribution mains. HDD was the method of choice in a few instances including a 2000 foot length to cross Park Hill Golf Course in northeast Denver. Certain-Teed Certa-Lok C900 Restrained Joint Pipe 8" diameter was used to cross the golf course in 4 separate pulls. The pits were placed in between fairways so that play could (when weather permitted) continue during construction. Overall, the Contractor saved money versus open cutting when factoring in savings realized because of reduction in surface restoration.

Figure 5 – Pipe laid out in the string method for HDD installation.

Identification of the Recycled Water Distribution System

Regulation No. 84, prepared by the Colorado Department of Public Health and Environment's Water Quality Control Commission, is the governing document regarding the use of reclaimed water in the state of Colorado. This document requires that piping used for reclaimed water be 'marked to differentiate reclaimed water from potable water or other piping systems'. In an effort to comply with this requirement, Denver Water specified the following items:

- Purple Manhole lids imprinted with the text, 'Recycled Water' and painted with a purple fusion bonded epoxy paint.
- Triangular valve boxes (as opposed to round valve boxes used for the potable water system) imprinted with the text, 'Recycled Water' and painted with a purple fusion bonded epoxy paint.
- Purple exterior on the pipe. For steel pipe this meant either a purple tape wrap or a purple polyurethane coating. Purple polyethylene encasement was used on ductile iron pipe. For PVC pipe, colorants were added to the manufacturing compound to make the pipe purple in color.
- Warning reading "CAUTION: RECYCLED WATER – DO NOT DRINK" installed along the spring line on both sides of the pipe. For steel pipe this generally was a vinyl sticker applied in the field. On ductile iron pipe, the warning was imprinted on the polyethylene encasement. For PVC pipe, the warning was imprinted directly on the pipe.
- Warning tape with the same message as above laid in the trench 1 foot above the top of pipe.
- Valve nuts were required to be purple and pentagon in shape, requiring special valve keys to be used.
- The opening direction for valves in the recycled water direction is specified to be counter-clockwise (left), opposite of that in the potable water distribution system.

Figure 6 – Photo of purple PVC pipe with warning.

Conclusion

By the end of Phase 2, over 90,000 feet of pipe will have been installed at a cost of approximately $40 million. Continued coordination with various public entities along with an ever improving public relations effort will be needed to successfully extend the recycled water distribution system to serve additional customers. Successful pipeline projects require as much effort dealing with outside forces as the design itself. Experience has shown that time spent during the design phase of the project coordinating, researching utilities, and talking to the general public and the proper permitting agencies will pay off in the form of fewer difficulties during construction.

References

D'Appolonia, E., Miller, L.E., and Ware, T.M., (1953), 'Sand Compaction by Vibroflotation", Proceedings, ASCE, Vol. 79 Separate No. 200, July 1953, pp. 1-23

Douglas, B.D. and Merrill, D.T. (1991). Control of water quality deterioration caused by corrosion of cement-mortar pipe linings. Denver: AWWA.

Colorado Department of Public Health and Environment – Water Quality Control Commission (2005). Regulation No. 84 Reclaimed Water Control Regulation. Denver, Colorado.

Jebe, W., and Bartels, K., (1983), "The Development of Compaction Methods with Vibrators from 1976 to 1982", Proceedings of the Eighth European Conference on Soil and Foundation Engineering, Helsinki, May 1983, Vol. 1, Sec 3.16, pp. 259-266

Challenges for Pipeline Bidding in a Seller's Market

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Introduction

The market for large diameter water pipeline construction in Texas has historically been a buyer's market compared to most of the remainder of the United States. Unit prices for pipeline construction in Texas have historically been very competitive, due to the large capacity of local contractors and pipe suppliers, low labor and material costs, and local construction practices.

In the last four years, pipeline construction demand in Texas has increased significantly due to growth and drought. Local demand has begun to exceed local supply and prices have increased as much as ninety-six percent in four years. Some of the price escalation is due to the increases in costs for steel, cement, gravel, diesel and other raw materials of construction. Contractors are also raising bid prices to cover material price volatility and to increase margins in this Seller's market. Other inflationary factors include increased labor cost due to reconstruction after Hurricanes Katrina and Rita and fewer bidders due to the bonding companies tightening up on the bonding capacity of general contractors.

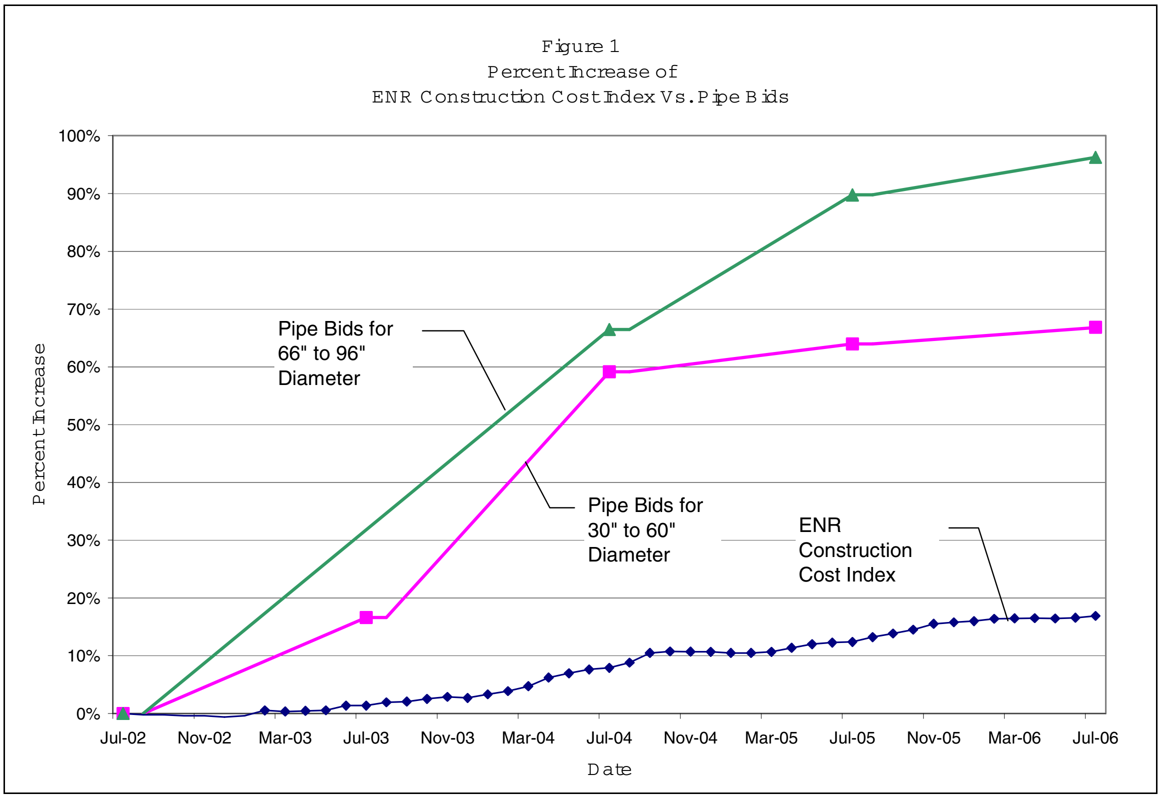
Measures used to mitigate the price escalation include structure of the bidding packages, allowing longer advertising periods, using smaller construction projects, recruiting more out of state contractors, allowing lower qualification requirements, allowing alternate pipe materials, and providing incentives.

This paper describes some of the escalation factors seen in the Texas market, measures taken to mitigate the price escalation, and lessons learned.

Historical Pipe Prices

Figure 1 shows a comparison of the Engineering News Record (ENR) construction price index versus pipe installation prices in Texas for the last five years. The cost for pipe installation is based on the cost per linear foot divided by the pipe nominal diameter for 57 projects. Separate curves are shown for medium diameter water lines (24-inch to 60-inch diameter) and larger diameter lines (72-inch to 108-inch diameter pipelines), since there is a larger increase in cost per diameter inch for the larger size range. This increase for the larger diameter range is for three reasons as follows:

1. The market for medium diameter pipe material is more competitive than larger diameter pipe. The medium diameter water pipelines are typically constructed of ductile iron pipe (AWWA C-151), bar wrapped concrete cylinder pipe (AWWA C-303), or steel pipe (AWWA C-200) in the Texas market. The larger diameter water lines are typically constructed of pre-stressed concrete cylinder pipe (AWWA C-301) or steel pipe.
2. The larger diameters are typically installed with larger construction equipment, due to the size and weight of larger pipe joints.
3. There are fewer qualified contractors available for the larger diameters, thus less competition at bid time.

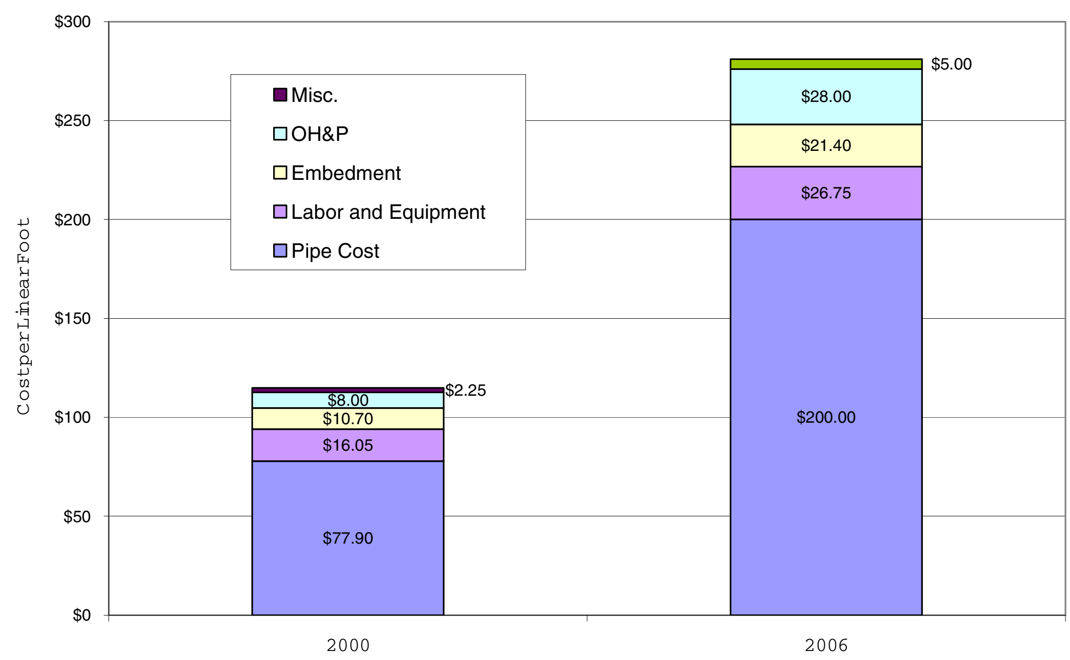


As can be seen in Figure 1, the inflation rates over the four year period are as follows:

- ENR index inflation rate has averaged 4.22% per year with a total inflation rate of 16.9% over the four year period.
- Medium diameter pipeline inflation rate has averaged 16.7% per year with a total inflation rate of 66.8%
- Large diameter pipeline inflation rate has averaged 24% per year with a total inflation rate of 96.3%

The market price for pipeline installation has clearly increased at a higher rate than the overall construction market, as measured by the ENR index. To understand why, we compared the unit prices for two 54-inch diameter pipeline construction projects. Project A was a 36-mile 54-inch pipeline which started construction in 2000. Project B was a 30-mile 60-inch and 54-inch pipeline which started construction in 2006. Both projects were in rural areas with little rock and were constructed with the same pipe materials. Figure 2 shows a breakdown of the various components of the pipe installation for 54" pipe. As can be seen, the largest increases are for pipe materials (250%), and the contractors' overhead and profit (40%).

Figure 2
Comparison of Pipe Installation Costs
2000 versus 2006 Projects



Inflation Factors

The inflation factors for pipeline installation can be broken into the following categories:

- Raw materials such as steel, cement, concrete, diesel, and gravel
- Labor
- Construction equipment

- Increased prices to cover price volatility, shortages, and other risks
- Increased profits due to market conditions
- More expensive design standards, such as pipe coatings

Figure 3 shows the price history for raw steel over the last five years. Steel prices were stable until 2004, but experienced a 60% increase through the summer of 2006. The world wide demand for steel, particularly in China, has fueled the increase in price. Steel prices are expected to decrease by 8.7 % in 2007 due in part to increased imports (ENR 12-18-06). Steel (or iron) is the most expensive material component in steel pipe and ductile iron pipe.

Figure 3
National Steel Prices ($/CWT)
Source: Engineering News Record

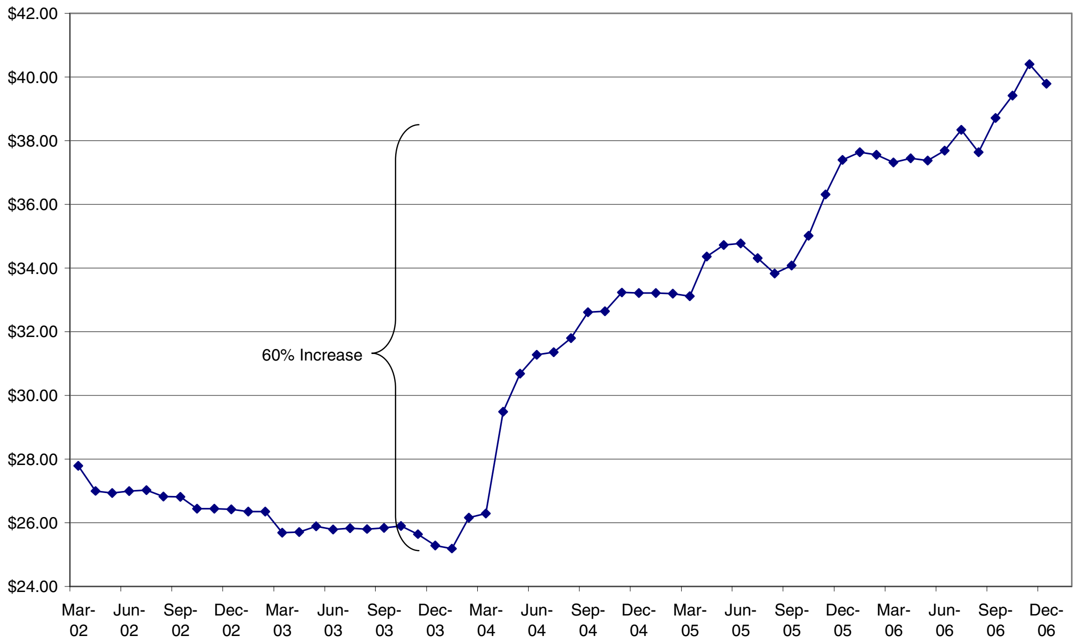


Figure 4 shows the price history for concrete and cement both nationally and in the Dallas, Texas market (ENR source). As can be seen, there was a steep price increase beginning in late 2005 in the national and Dallas market. Prices of cement have begun decreasing in the summer of 2006. A recent surge in imports from China has helped to eliminate shortages and prices are starting to level off. Overall prices remain high and prices are expected to increase nationally by 3.3% in 2007 (ENR 12-18-06). However, prices could increase by higher rates in hot markets such as Texas. Overall, cement and concrete are a minor factor in the total cost of pipeline installation.

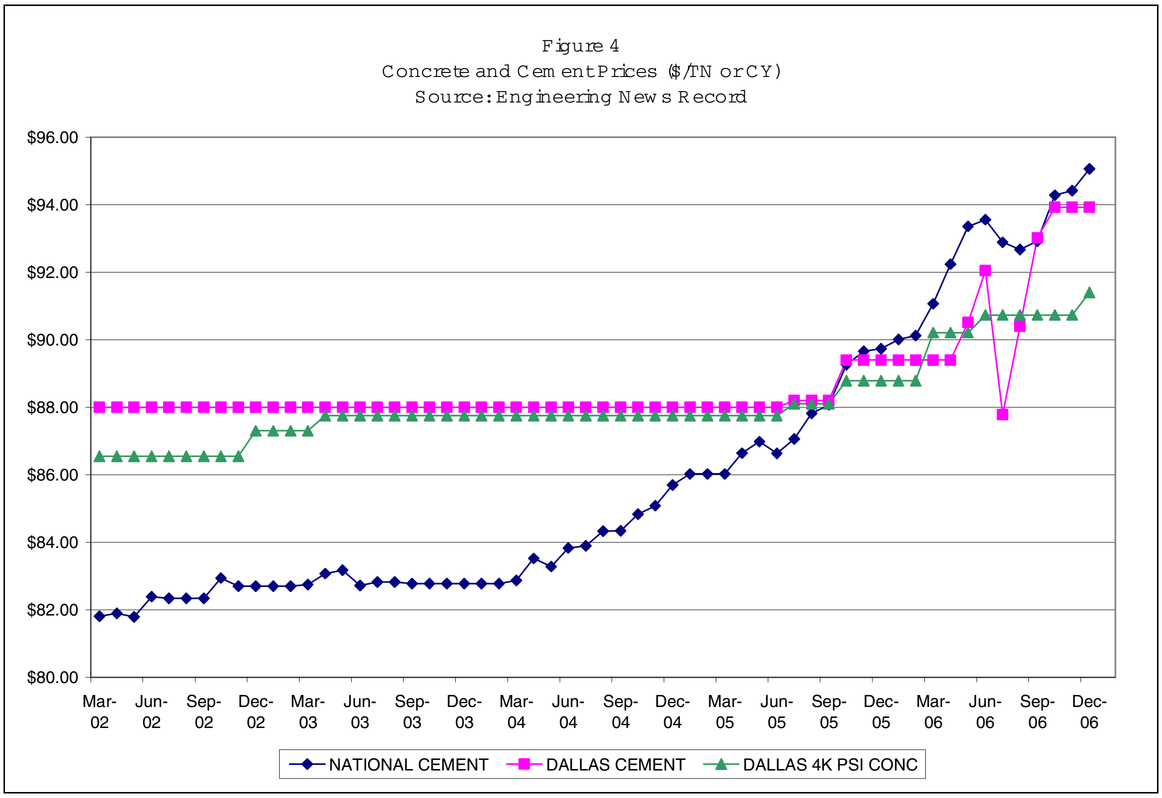


Figure 5 shows the price history for diesel fuel (source: U.S. Dept of Energy). As can be seen, there has been a 220% increase in price from January 2002 to August 2006, followed by a 22% decrease at the end of 2006. The effect of diesel price increases is difficult to predict for pipeline installation, since it affects almost all components of the construction to some extent. Diesel prices affect hauling costs for material deliveries as well as operating costs for construction equipment. The cost of fuel to operate construction equipment is estimated to be one to three percent of the total costs. This amount does not include diesel for hauling of materials to and from the job site.

Imported gravel is a common embedment material for pipeline installation in Texas. Gravel prices are heavily influenced by local availability and hauling costs. Gravel prices are estimated to have increased by 9 percent from 2002 through the end of 2004, with an increase of 19 percent from 2005 through late 2006 (Source: Bureau of Labor Statistics). Overall, gravel embedment material is estimated to be 4 to 8 percent of the total pipeline construction costs.

Figure 6 shows the national trend for skilled and common labor over the last five years. As can be seen, there has been an overall increase of approximately 18% over the period. Labor prices are expected to increase by 5.0% in 2007 (ENR 12-18-06). Overall, the construction labor costs are estimated to be 3 to 8 percent of the total pipeline construction costs.

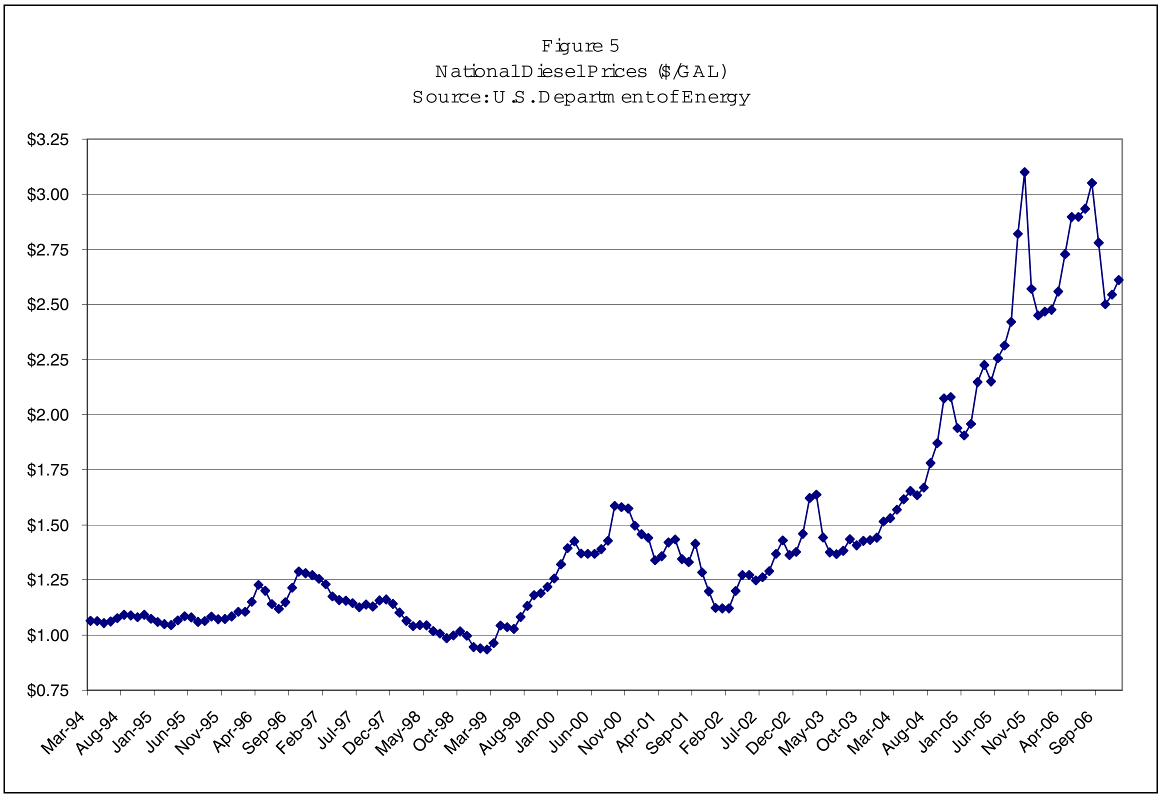


Figure 5
National Diesel Prices ($/GAL)
Source: U.S. Department of Energy
$3.25
$3.00
$2.75
$2.50
$2.25
$2.00
$1.75
$1.50
$1.25
$1.00
$0.75
Mar-94
Aug-94
Jan-95
Jun-95
Nov-95
Apr-96
Sep-96
Feb-97
Jul-97
Dec-97
May-98
Oct-98
Mar-99
Aug-99
Jan-00
Jun-00
Nov-00
Apr-01
Sep-01
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Nov-05
Apr-06
Sep-06

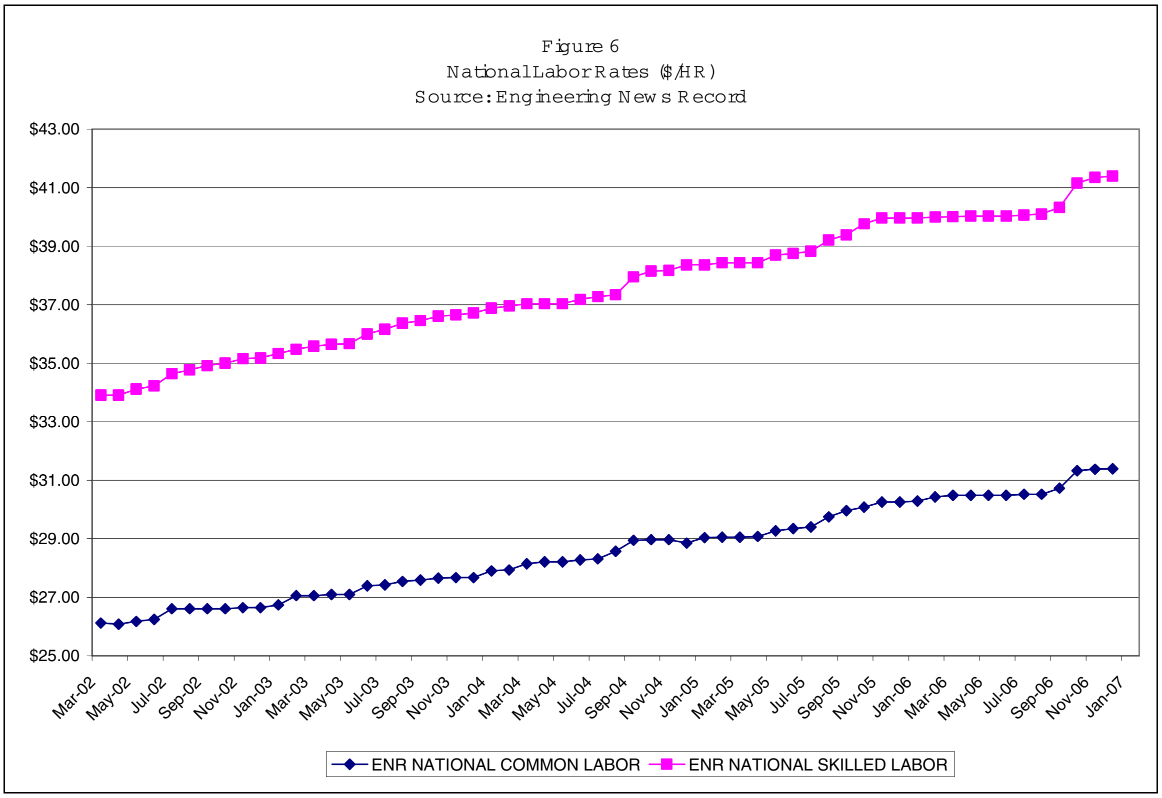
Figure 6
National Labor Rates ($/HR)
Source: Engineering News Record
$43.00
$41.00
$39.00
$37.00
$35.00
$33.00
$31.00
$29.00
$27.00
$25.00
Mar-02
May-02
Jul-02
Sep-02
Nov-02
Jan-03
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ENR NATIONAL COMMON LABOR
ENR NATIONAL SKILLED LABOR

Construction machinery and equipment does not appear to be a major culprit in pipeline construction inflation. Equipment prices have increased only 17% in the last five years (source: Bureau of Labor Statistics).

Contractors face many challenges in today's market. Which subcontractors and suppliers will not honor their price quotes after bid time? Will deliveries of materials be on time? Will the delays affect the construction schedule and costs? Will shortages of materials require work stoppage or re-design of the facilities? Will the owner negotiate changes due to material delivery delays or shortages? Many contractors have indicated these are huge factors in today's bid prices, and smart contractors are putting in larger risk premiums on pipeline projects. These factors sometimes result in large swings in bid prices from one project to another. These inflationary factors will probably continue until supply catches up with demand and material prices stabilize.

The final factor in pipeline construction inflation is old-fashioned American capitalism. When demand exceeds supply prices go up. When prices go up, more supply becomes available and the market corrects itself. Nationally, the US Department of Commerce reports an 11% increase in water supply construction and a 19.6% increase in sewage and waste disposal construction. Unfortunately for owners (and fortunately for contractors), there has been a surge in demand for pipeline construction in Texas. This increase in demand is due to population growth in Texas, drought, and pent up demand after 9/11. Also, at least four large pipeline contractors left the North Texas market in the last four years. Some of the remaining contractors report that the bonding companies are tightening up on the size of contracts they can bid. These factors have resulted in fewer bidders for a larger volume of pipeline construction. Contractors can monitor the status of future projects and bid projects with higher profit margins, since they know there will be other chances to bid projects. These inflationary factors will probably continue until supply catches up with demand.

Another factor that should not be overlooked in evaluating pipeline construction inflation is the evolution of more expensive design standards. Industry practices continue to become more expensive to address past pipeline failures and to meet higher goals of pipeline life expectancy and lower risk. These factors can affect specifications for pipe material options, coatings, cathodic protection, joints, quality control testing, and many other components. These factors should not be considered inflation, but should be categorized as increases to meet higher performance standards.

Inflation Mitigation Techniques

Owners and engineers may take several approaches to mitigate the effects of inflation. These include making modifications to the typical contract structure and construction contract general conditions as well as making changes to the design and cost estimating procedures.

The following changes to a typical contract structure could improve the bidding conditions and help reduce the cost of a large water transmission project.

- **Bid smaller pipe sections.** As mentioned previously, the bonding capacity of many contractors is being tightened or reaching its limit due to the increase volume of work. Some contractors may choose to not bid or may be unable to bid a long, expensive pipeline project. One solution is to break pipelines into shorter segments. This allows more contractors to bid on the segments. A recent 50-mile, 84-inch pipeline was broken into four pipeline segments. Similarly, a 30-mile, 54/60-inch pipeline was broken into three segments. For both projects, all pipeline segments received at least four bidders on each pipe segment with prices coming in under budget.
- **Allow joint ventures for pipe suppliers and contractors.** One way for contractors and manufacturers to compete for jobs when their bonding capacity is limited is to partner with another firm. In some cases, pipe manufactures that are stretched too thin have combined forces with other suppliers to compete on projects. Historically, our firm has not allowed multiple pipe suppliers on a contract, but recently allowed this on a 12-mile 84" pipeline to try to encourage competition.
- **Employ the "Winner Takes All" approach.** Another option available to owners and engineers is to bid multiple contracts at one bid opening and allows contractors to offer a deduct if a contractor is awarded all phases of the project. When projects are broken into smaller segments, bidding the projects at the same time may entice larger contractors to bid in a market in which they may not typically be drawn. One company may be able to do the entire job more efficiently with less overhead and pass those savings onto the owner. It also encourages contractors to sharpen their pencils compared to bidding the project in sequential phases, since there will not be a second chance at another portion of the project. There are some potential drawbacks to this bidding structure. Smaller contractors who may not have the bonding capacity to build the entire project may only be able to bid one segment, thereby reducing competition on other segments. The winner takes all provision worked well on our 156-mile 60"/53" Ivie Pipeline Project, which received nine bidders during a 1992 buyer's market. However, the winner takes all approach was not as successful on the 20-mile 96"/84" Eagle Mountain Project, which received only four bids on one section and two on the other section. For these reasons, the winner takes all strategy may be best employed in a buyer's market.
- **Be realistic in your contract's construction schedule.** It is important to allow enough time for construction, assuming one pipe manufacturer and one pipe-laying crew. Tight construction schedules tend to reduce the number of bidders and may limit some pipe manufacturers from bidding. Allowing longer construction periods may also enable a contractor to bid multiple pipe

sections instead of having to focus on making a tight schedule on one segment.

- **Use the A+B bidding equation.** In some instances, meeting an aggressive schedule is paramount to the success of the project. A + B bidding requires the contractor to submit (A) the amount bid and (B) the days it will take to complete the bid. The calendar days required to complete the project is multiplied times a daily value (usually equal to the liquidated damage) to get the B portion of the bid. The A and B are added together to evaluate the bids. A+B bidding, in conjunction with an incentive payment for early project completion, was used successfully on a recent pump station project. For pipeline construction, A+B bidding gives the contractor an incentive for streamlining their production and lets the contractor set the fastest possible schedule.
- **Pre-qualify.** In a seller's market, contractors can pick and choose what projects they go after. In some cases, contractors may not bid a project if they're concerned about meeting the minimum qualifications. But, if they're pre-qualified, they can be confident of getting the award if they're the low bidder.
- **Benchmark material prices at bid time.** Recent history has shown wild swings in the prices of steel, concrete, copper and fuel. Many contractors have been burned by rapid price escalations that occur post-bid. During these times, contractors tend to add money to their bids to cover the risk of material escalation. One way to offset this is for the owner to take on some of the risk. This can be accomplished by setting a benchmark for material prices at bid time with the owner either paying for rising prices above the benchmark or receiving a deduction for prices falling below the benchmark.

In addition to adjusting the contract structure, sometimes making small changes to the general conditions can make a project more attractive to potential bidders. The general conditions will spell out how a Contractor will be paid for the work. Many times, full payment for pipeline installation will have conditions such as completing the hydrostatic test or establishing grass on the right-of-way. Five to 10 percent of the payment may be withheld even if the cost of these items is closer to 1 percent of the total cost. Similarly, excessive retainage can put a heavy burden on the Contractor's cash flow. Lightening up on the retainage and other withholdings may help the bottom line at bid time and attract more bidders. A few contractors have indicated they would not bid projects because the payment provisions caused cash flow problems.

Another way to mitigate inflation and lower bid prices is for owners to share in the risk of other portions of the project. The amount of rock excavation on a pipeline project is one area of risk that typically falls to the contractor. An owner who pays for thorough geotechnical investigation during the design phase can reduce the amount of risk the contractor must take on.

In one case, a project was re-bid after a lack of bidders resulted in poor bids. One contracting firm revealed that they didn't bid because of a concern that the pipe delivery trucks would have to use sub-standard county roads, requiring them to add money to the job for repairing these roads. When the project was re-bid, the general conditions limited the potential repair areas to roads directly parallel with the pipeline route, so long as the contractor used legal loads, thus limiting the contractors risk and reducing bid inflation.

Another way to mitigate rising costs due to inflation is to pay close attention to some of the pipeline details. Allowing techniques that increase contractor's productivity can reduce costs. For example, allowing weld after backfill will increase pipe laying productivity by taking joint welding out of the critical path. Imported pipeline embedment can be a very large expense for contractors. Allowing multiple embedment options such as trench excavated material or cement-stabilized trench excavated material can result is a large savings to the contractor that may be passed on to the owner. Also, allowing the contractor to spoil excess material over the pipeline right-of-way can be a large cost saver, particularly for large diameter pipelines. Finally, allowing competitive pipe material options results in more pipeline manufacturers to compete for the project resulting in lower prices.

Once you've done everything in your power to mitigate inflation, the next step is to keep a close ear to the ground to reduce sticker shock at bid time. In a seller's market, accurate cost estimating become more difficult and requires more diligence. Engineers would be wise to follow these suggestions:

- **Stay up on the market.** Changes to the market can drastically affect material prices almost overnight. Follow the major markets that affect pipeline prices such as steel, concrete and fuel prices and make changes to the estimate accordingly.
- **Get contractor and supplier input.** Contractors and suppliers must keep close tabs on the marketplace. Use your contacts to review your cost estimates, get updated pricing, and to learn about other issues you may have missed affecting prices.
- **Update estimates frequently**. Things change rapidly. An estimate performed at the 50% level can change drastically by bid time. In the beginning of a project, set a reasonable schedule for updating cost estimates, perhaps once a month, and update the client as changes occur.
- **Adapt design to market conditions where possible.** Pipe material that appeared too expensive one month might become more attractive later. Perhaps resin shortages raised the cost of PVC pipe to match steel or concrete pipe. Pipe manufacturer's backlog and delivery schedule can greatly impact bid prices. Be flexible.
- **Advertise smart.** Consider advertising for a full five weeks to allow plenty of time for contractors to prepare their bids. Allow adequate access to the pipeline route and schedule test digs if required. Be proactive about

communicating the invitation to bid through faxes, email and website notifications. Also, carefully choose the bid date so that it doesn't conflict with other bids or industry conferences.

Finally, remember, your reputation precedes you. Projects will attract more bidders if the owner or engineer has a reputation for being fair and responsive. Engineers should be responsive on submittals and consistent on enforcement of the specifications. Inspectors should enforce the specifications to achieve a quality project, but be fair. Owners should make payments in a timely manner and negotiate in good faith on change orders.

Conclusions

The Texas market has seen a large increase in pipeline construction costs over the last four years. Several factors have contributed to this increase including growing demand, fewer contractors and material price volatility. It is anticipated that overall pipe installation prices should escalate 6 to 8 percent in 2007.

In a seller's market, it is important for the Owner and engineer to use diligence in assessing the market, updating cost estimates, and taking steps necessary to mitigate large cost increases to avoid sticker shock at bid time.

References

Engineering News-Record, McGraw Hill Construction. Multiple Issues, 2005-2006. Reprinted with permission.

U.S. Department of Energy, Energy Information Administration, www.eia.doe.gov.

U.S. Department of Labor, Bureau of Labor Statistics, www.bls.gov.

Pipeline Planning and Design Considerations in a Challenging Urban Environment

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Abstract

A contamination plume in the public water supply aquifer of the Town of Wilmington, Massachusetts threatens five of the Town's nine water supply wells. In early 2003, the Town took those five wells off-line indefinitely, which created a significant water supply deficit. The Town recently completed a Comprehensive Water Resources Management Plan that investigated various options for the Town to augment its water supply. The Plan identified constructing a permanent connection to the Massachusetts Water Resources Authority (MWRA) regional water system as a sound, long-term water supply strategy.

Although the MWRA water system is located within only 3.2 km (2-miles) of the Wilmington town-line, in north Woburn, potential project routes are constrained by an active commuter rail and an inactive landfill on the west, Interstate-93 on the east, and a busy industrial area that contains the "Industri-Plex" CERCLA (Federal Superfund) Site in between.

Past uses of the Industri-Plex Site include chemical manufacturing and manufacturing of glue from animal hides. By the late 1990's, approximately half of the Site was covered with a protective cap to prevent exposure to contaminated soils and portions of the Site were redeveloped. Investigations into the extent of the groundwater contamination and residual migration continue.

In December 2002, the Town began investigations to determine alternative routes for the proposed water transmission main connection. Between 2002 and 2005, the Town coordinated extensively with numerous parties to gain an understanding of the limitations of this area and to identify the most feasible project route. The selected route proceeds through municipal roadways and a remediated portion of the Industri-Plex Site to reach the MWRA water system. The proposed pipeline installation within the Site required careful design considerations and various construction techniques to prevent disturbance to the protective cap, keeping in mind an overall goal of minimizing disruption to abutting businesses during construction.

Background

The municipal water supply system in the Town of Wilmington, Massachusetts consists of nine permitted water supply wells, two water treatment plants, three storage tanks, and over 193 km (120-miles) of distribution main. Five of the water supply wells are within the Maple Meadow Brook Aquifer (MMBA), which encompasses much of south-central Wilmington. There is a significant amount of industrial land use in south-central Wilmington, which makes the MMBA wells potentially susceptible to groundwater contamination.

The Source Water Assessment Program (SWAP) report, completed in January 2002 by the Massachusetts Department of Environmental Protection (MA-DEP), indicated the Town of Wilmington has a high susceptibility for contamination of their aquifers due to the land uses in these areas. This SWAP report had identified "high threat" land uses, where the potential for spills and releases of oil and hazardous materials are elevated.

Existing water supply wells within the MMBA have historically shown increased levels of ammonia. Although ammonia is not a regulated contaminant, it can be converted to nitrate and nitrite. Both nitrate and nitrite have established drinking water Maximum Contamination Levels. Because of the historic contamination threats and water supply limitations, Wilmington began the regulatory and political process to gain approval for emergency water supply from the Massachusetts Water Resources Authority (MWRA) regional water system. This approval would provide Wilmington with access to a high quality, reliable emergency water supply. On September 18, 2000, the Town requested approval from the MWRA for emergency water supply. In October 2000, the MA-DEP sent a letter to the MWRA supporting Wilmington in their request. On September 18, 2001, the MWRA Advisory Board approved the Town of Wilmington's request, allowing the Town access to emergency MWRA water supply.

In early 2003, the discovery of the carcinogen N-nitrosodimethylamine (NDMA) in the aquifer and four water supply wells forced the Town to discontinue use of all five wells in the MMBA, and prompted the MA-DEP to declare those wells unfit for drinking water purposes indefinitely. At that time, the Town sought and was granted a declaration of water supply emergency from the MA-DEP, which allowed an emergency water supply withdrawal from the MWRA to satisfy immediate water supply needs. The Town utilized MWRA water supply through an emergency water system interconnection with the City of Woburn, an adjacent community that is partially served by the MWRA water system.

Although other parties continue to investigate treatment of the NDMA contamination as an option to restore the MMBA, those wells cannot be considered a reliable long-term supply, and the Town must replace that lost capacity. Wilmington recently completed a Comprehensive Water Resources Management Plan that investigated various options for the Town to augment its water supply. The Plan identified a full-time water supply from the MWRA water system as a sound, long-term water supply strategy. Wilmington's water system interconnection with Woburn has a limited capacity; therefore, a permanent pipeline connection to the MWRA water system is now a priority for the Town. The Town is also now pursuing approval to become a full-time member community of the MWRA water system.

Coordination and Planning Challenges

After receiving approval for emergency MWRA water supply in 2001, Wilmington began planning a pipeline connection to the MWRA water system. The Town investigated alternative routes for the water transmission main in 2002 and 2003. The MWRA water system terminates approximately 3.2 km (2-miles) south of Wilmington, in north Woburn; however, potential pipeline routes are constrained by an active commuter railway and an inactive landfill on the west, Interstate-93 on the east, and a busy industrial area that contains the "Industri-Plex" CERCLA (Federal Superfund) Site in between, which made route selection difficult.

Past uses of the Industri-Plex Site include chemical manufacturing and manufacturing of glue from raw animal hide and chrome-tanned hide wastes. By the late 1990's, approximately half of the Site was covered with a protective cap to prevent exposure to contaminated soils and portions of the Site were redeveloped for a multi-modal Regional Transportation Center, Interstate-93 interchange, public road extension, Target Stores, and Metro-North Office Park including Raytheon and Residence Inn – Marriott. Investigations into the extent of the groundwater contamination and residual migration continue.

Based on initial investigations, traversing the Industri-Plex Site appeared to be the most feasible and direct pipeline route. The Town met with parties involved in the management and remediation of the Industri-Plex site in 2003 to discuss the potential for installing a water transmission main in existing roadways that cross the Site. These roadways are referred to as Commerce Way and Presidential Way in Woburn. The parties involved include the Environmental Protection Agency (EPA), the MA-DEP, the Industri-Plex Remedial Trust, the Industri-Plex Custodial Trust, and the City of Woburn. From these meetings, the Town learned that construction within the Site is subject to strict obligations and requirements for compliance with federal superfund laws and that planning, design, and construction activities would face intense scrutiny and require close coordination with the key parties referenced above. Although there were no indications that pipeline construction within the Site was not feasible, other potential routes were investigated to ensure that less complex options were not available.

Alternative routes included utilizing: municipal roadways in the Town of Reading to the east; the northbound and southbound sides of the Interstate-93 right-of-way to the east; and several routes within the City of Woburn.

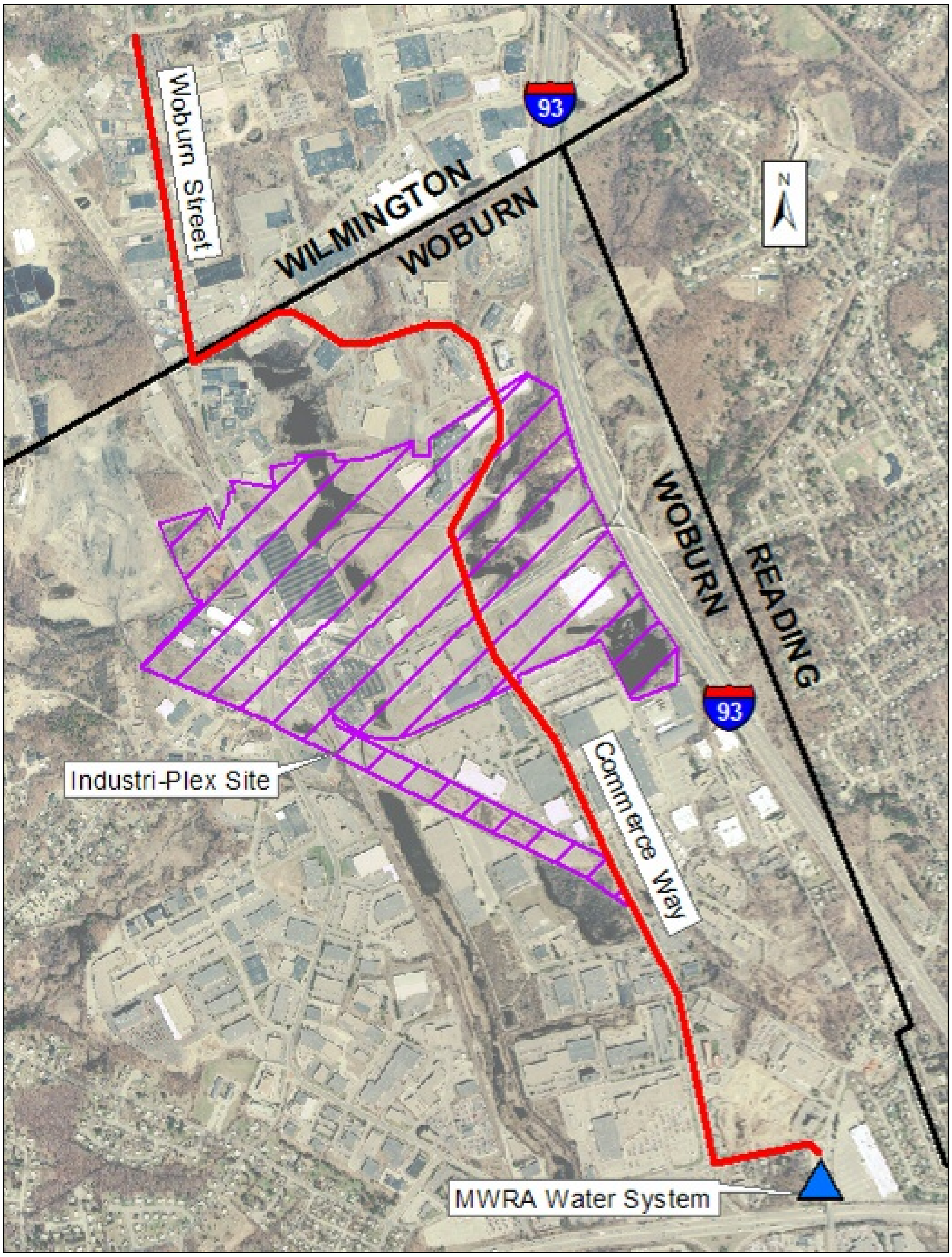
From 2003 to 2005, Wilmington evaluated the technical feasibility, cost, and environmental impacts of the various alternatives on Woburn, Reading, Wilmington, and the Industri-Plex Site. Coordination with additional parties was required, while the City of Woburn, the EPA, and the Industri-Plex Trusts were involved throughout the evaluation process. Many of the alternatives posed greater limitations than the proposed route through the Industri-Plex Site, such as legal challenges, access restrictions, and contamination from sources other than the Site. The Town ultimately concluded that the proposed route through the Site was the most technically and economically feasible route, and that the use of careful design considerations and various construction techniques would protect the Site and minimize disruption to the surrounding environment. Figure 1 depicts the proposed pipeline route.

Design Challenges

Design of the water transmission main along the selected route began in 2005. The project consists of approximately 3,780 meters (12,400 linear feet) of 510 mm (20-inch) diameter ductile iron water main and a MWRA meter vault in the City of Woburn. It also includes installation of approximately 945 meters (3,100 linear feet) of 405 mm (16-inch) diameter ductile iron water main and a Town meter and valve vault in the Town of Wilmington. In addition to water main installation, the work includes the installation of fittings, valves, hydrants, and water services. Approximately 915 meters (3,000 linear feet) of water main construction is within the boundaries of the Industri-Plex Site.

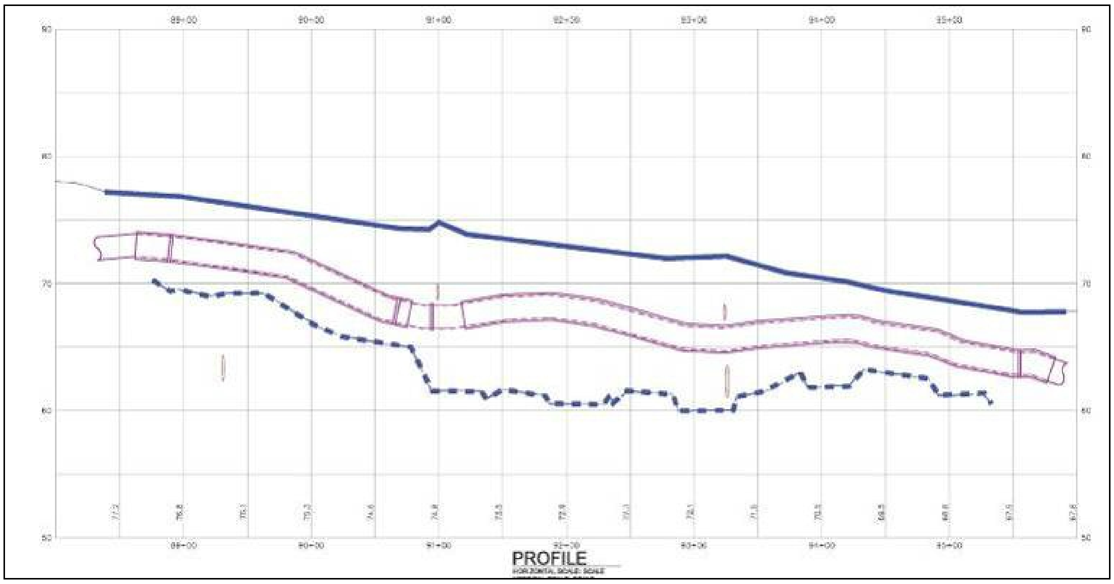
The greatest challenge of the project design was to ensure that construction of the pipeline does not disturb the heavily-contaminated soils located below the protective cover, which consists of a permeable geotextile fabric. The cover was suspected to be located at a shallow depth below the existing roadway in the Site and the regulations governing the Site require all excavation proposed within 305 mm (12-inches) of the cover to be conducted by non-mechanical methods. Hand excavation would not be an economical construction method for this portion of the project. A utility corridor was constructed in the southbound lanes of the roadway during remediation to allow utilities to be installed within clean soil; however, there is no space remaining in the corridor. The Site was documented extensively during the remediation activities in order to meet the EPA requirements for record drawings; therefore, the Town was able to identify the depth of the protective cover from surveyed points at regular intervals and from the survey of the existing roadway. After a close review of the record drawings, it was discovered that the cover was installed at a greater depth than anticipated near the center of the roadway to accommodate a 1,220 mm (48-inch) diameter reinforced concrete drain pipe that was installed in the roadway median.

Figure 1


Woburn Street
93
WILMINGTON
WOBURN
N
WOBURN
READING
93
Industri-Plex Site
Commerce Way
MWRA Water System

The design engineer determined that the pipeline could be installed in the northbound lanes of the roadway at a shallow depth while maintaining a 305 mm (12-inch) separation from the protective cover. A profile showing both the elevation of the protective cover and the roadway surface was prepared to assist with the pipeline design. In order to maintain a 305 mm (12-inch) separation from the cover plus some additional separation for contingency purposes, the depth of bury for the pipeline was as shallow as 0.76 meters (2.5-feet) in some locations. The design included preinsulated water main to protect the pipeline from freezing along most of the installation above the cover. Figure 2 illustrates the profile prepared for the design drawings. The dashed line represents the protective cover and the solid line represents the roadway surface. The pipeline elevation was adjusted to clear several drain crossings, while maintaining at least 455 mm (18-inches) of separation from the cover at the nearest point.

Figure 2



Upon completion of the design, there was a concern that the elevation of the protective cover could vary substantially between the surveyed points. Consequently, a method to check the accuracy of the record data within the Industri-Plex Site was included in the contract documents. The design requires the selected contractor to conduct exploratory excavation prior to construction along the pipeline alignment in areas above the protective cover utilizing a non-destructive air-vacuum excavation method. The excavation method involves removing the surface material over approximately a 305 mm (12-inch) by 305 mm (12-inch) area at designated locations utilizing compressed air jets that simultaneously loosen soil while a vacuum extracts the debris. The documented fill material below the roadway is clean sand, which is ideal for this unique excavation method. The contractor is required to conduct vacuum excavation to the proposed trench depth shown on the drawings to verify the depth of the protective cover. If the cover is encountered, this excavation method will not damage the cover, and will allow for a modification to the proposed pipeline elevation prior to pipeline trench excavation.

Other unique design provisions include providing air and vacuum release valves at appropriate locations along the pipeline route, below grade flushing hydrants, and isolation valves at regular intervals and at key locations in the Industri-Plex Site to minimize disruption if a water main break were to occur. An interesting aspect of the project is that portions of roadways within the route, Commerce Way and Presidential Way, were extended after remediation of the Site and had not yet been accepted as public ways by the City of Woburn. Wilmington realized that the acceptance of these roads by the City was critical prior to construction; otherwise, the Town would need to acquire easements by private landowners to allow installation of the pipeline. Since the easements would be difficult to obtain, Wilmington coordinated closely with the City to ensure the acceptance would occur.

Although the proposed excavation and pipeline installation is not anticipated to impact heavily-contaminated soils under the protective cover, a significant amount of effort is required by the contractor to manage activities in this site and coordinate with the EPA and the MA-DEP. Approximately 305 meters (1,000 linear feet) of water main installation in Woburn will occur in a state roadway, which is subject to stricter construction standards and must be performed overnight. Another area of the project within the Industri-Plex Site that includes the Interstate-93 interchange will require work to be performed overnight. Surface restoration will consist of backfill, compaction, and trench pavement; however, the City of Woburn requires overlay pavement restoration over the entire width of the roadway in addition to trench pavement within a majority of the impacted roadways. A resident engineer will be on-site throughout construction to ensure that construction activities conform to the project specifications and that the progress of work adheres to the project schedule. In addition, a Licensed Site Professional with the state of Massachusetts will be on-site during the work located within the Industri-Plex Site.

Bid Results

The project was bid in October 2006, approximately 14-months after the design was completed. During this time, the project was under review by the City of Woburn, the EPA, and the Industri-Plex Trusts. Wilmington and Woburn also needed time to finalize an acceptable intermunicipal agreement for the project. In addition, Wilmington was required to obtain a "Grant of Right" for construction in the Woburn public way prior to construction. The fact that a portion of the route was within a roadway that had not yet been accepted as a public way by the City complicated this effort.

A total of ten bids were received for the project. Table 1 summarizes the results of the bid.

Table 1

Engineer's Estimate	Low Bid	Average Bid	Median Bid
$4,016,461	$3,465,585	$4,138,286	$4,184,035

The Town has awarded the project and construction is anticipated to begin in the spring of 2007 and be complete by December 2007.

Conclusion

The design process took four-years to complete from the planning phase to the bid opening. The completion of the project design is a result of cooperation and collaboration between many parties and several years of diligent effort. The Town of Wilmington faced many issues during this period, several of which were significant enough to potentially prevent the project from progressing to construction. However, the Town remained persistent and considered a wide range of alternatives and unique construction methods to resolve the challenges that arose.

The Town of Wilmington will continue to withdraw MWRA water supply through its emergency interconnection with the City of Woburn water distribution system during periods of high water demand until the project is completed. Once complete, the pipeline will serve as Wilmington's permanent connection to the MWRA water system.

How a directional drilled river crossing water main saved significant permitting requirements, construction time and money, Aroostook River, Caribou, Maine

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Abstract

Utilizing horizontal directional drilling for the installation of a river crossing water main saved the Caribou Utilities District in Caribou, Maine significant permitting requirements, construction time and money.

The Caribou Utilities District in Caribou, Maine, recently transitioned from a surface water source to a new groundwater source.

The District retained the services of Wright-Pierce of Topsham, Maine, to design the wells, pump station and transmission mains. A 12-inch ductile iron water transmission main was used to bring water from the wells and pump station to the distribution system, for the entire length except for a required river crossing, which was installed by horizontal directional drilling (HDD).

This paper will address processes by which HDD was selected as the best alternative for the river crossing, the permitting benefits of selecting HDD, problems encountered during construction, and the lessons learned from this interesting project.

Introduction

With an aging surface water treatment facility unable to comply with current and future drinking water regulations, the Caribou Utility District in Caribou, Maine determined that developing a new groundwater supply was the best alternative to improve water quality and meet regulatory requirements. After extensive groundwater investigations, an appropriate groundwater source was located in a gravel deposit adjacent to the Aroostook River. The locations of the new wells were about a mile north and on the opposite side of the river from the service area in the city of Caribou. Water from the new wells and pump station needed to be transmitted a mile south and across the river to feed the existing distribution system.

The District retained the services of Wright-Pierce of Topsham, Maine, to design the wells, pump station and transmission mains. A 12-inch ductile iron water transmission main was used to bring water from the pump station to the distribution system, for the entire length except for the river crossing. The river crossing, which was installed by directional drilling, saved the District extensive permitting requirements, time and money.

The Project

Permitting

The District's conventional filtration water treatment facility, which had been in operation since the 1940's, supplying water to the City of Caribou, a community in Northern Aroostook County, Maine, was not able to treat the water to meet current drinking water regulations. The District, which adds chlorine to the water as a disinfectant for microbial protection, had problems with Disinfection By-Products (DBP's) created when the chlorine reacted with the treated surface water. The presence of these elevated levels of DBP's caused the Maine Department of Health and Human Services (DHHS) to issue a consent order to the District outlining requirements and deadlines by which the District must solve the DBP problem.

The project, which was partially funded with state and federal monies, required that a preliminary alternatives analysis be performed to determine the best alternatives for improving the water quality and meeting drinking water regulations. Since renovation of the existing surface water treatment plant was a very expensive option, the District performed extensive investigations of alternative groundwater sites to find a location that met the water quantity and quality requirements, was within a reasonable distance to pipe to the distribution network, and could be reasonably protected as a source. A water source was located and acquired adjacent to the Aroostook River to the north of the City.

The new source needed to be connected to the distribution network. To meet the timeline required by the DHHS consent order, the permitting process for crossing the river could not be a lengthy process and needed to meet the needs of the State Regulator's, the District's needs, and minimize potential environmental impacts. Options for crossing the Aroostook River were evaluated. At a location about a mile south of the new wells, the District had an existing sewer force main crossing of the river. This location, where the District already owned the property on both sides of the river, appeared to be the ideal crossing location.

At the location of the proposed crossing, the Aroostook River was designated by the State of Maine as an *Outstanding River Segment*. This offered special protection of the river segment under Maine Statutes. This triggered an automatic Natural Resources Pre-Application Meeting with the Maine Department of Environmental Protection (MEDEP) to examine the project and determine that it is designed to avoid and minimize environmental impacts. At this early meeting, MEDEP strongly recommended that directional drilling be utilized to complete the river crossing and minimize the environmental impacts to the river.

In addition to the *Outstanding River Segment* designation, the river is home to a significant wild brook trout population under special regulation for quality brook trout management from the Maine Department of Inland Fisheries and Wildlife (MEIF&W) and is home to the pygmy snaketail dragonfly *(Ophiogomphus howie)* a threatened species of dragonfly. To protect the trout and the dragonfly larvae, the State would have likely placed restrictions on open cut disturbance of the river that would have had significant impact on the construction time-line, project cost, and the District's ability to meet the consent order deadlines.

As part of the preliminary engineering evaluation, the project site was evaluated for wetlands. Both embankments of the river were mapped as wetlands. If these wetlands were disturbed, it triggered another permit that would need to be obtained.

Preliminary recommendations by the MEDEP were as follows;

- Investigate directional drilling as viable option for installation of the river crossing.
- If directional drilling is a workable alternative, and the drilling and receiving pits with associated ground disturbance, could be kept a minimum of 50-feet from the river bank, **no** MEDEP permits would be required. This ruling was based on the assumption that the riverbed and the wetlands on the embankments would not be disturbed by drilling under the river.
- If directional drilling is not feasible and the District wishes to pursue the water main crossing by open cut method, the following would be required;
    - Full Natural Resource Protection Act (NRPA) Permit application submission. This is a detailed permit that would have required public comments and potentially public hearings that could have added significantly to the project design schedule.
    - Development of full plans and specifications for an open cut river crossing utilizing portable dams and dewatering.
    - Incorporation into the documents of any special provisions or resource restrictions required to protect the fishery and threatened species.
    - Strict supervision of the construction by the MEDEP.

Preliminary Investigations

Based on the MEDEP recommendations, the District proceeded with preliminary geotechnical investigations to determine if directional drilling would be possible at the proposed crossing locations. Discussions with several experienced HDD contractors indicated that test borings directly adjacent to the proposed crossing location would provide the best information on viability for directional drilling. They stressed that the test borings not be installed directly along the proposed route as this could create possible conduits for the drilling slurry to surface during the installation.

Visual observation of the river bottom downstream of the crossing location indicated exposed bedrock outcrops and the contractors had concerns regarding the depth of the overburden at the proposed location. Also, the overburden consists primarily of glacially deposited, well graded coarse gravels, and the contractors expressed concerns about "frac-out" of the drilling slurry during the high pressures of pull-back of the pipe if it was not installed deep enough.

Test borings were completed on both river banks and along the riverbed at a 20'-25' offset to the proposed pipeline route. The borings ranged from 30-feet to 60-feet deep and indicated that the overburden was of sufficient depth to install the water main by HDD.

Water Main Crossing Design

Upon confirmation of feasibility to install the water main by HDD, the District directed Wright-Pierce to develop the design for the crossing.

The District, which was on a very tight financial budget, could not afford to install redundant pipelines across the river, so a reliable solution needed to be found for a single pipe crossing.

To maximize the flow and minimize hydraulic restrictions, 16-inch SDR-9 DIPS HDPE pipe was specified. This pipe most closely matched the inside diameter of the 12-inch ductile iron water main from the new pump station to the distribution system. Thick walled pipe (SDR-9) was specified to withstand the high static pressures in excess of 150 psi that would occur in the river crossing segment, now the lowest point in the distribution system. Also, thermal butt-fused pipe eliminated pipe joints under the river that could potentially create future leaks.

The most significant hurdle to the design of the crossing was the very steep 50-foot high embankment on the west side of the proposed route from the rivers edge up to River Road where the water main continued to the new pump station. The proposed route is shown on the plan in Figure 1.

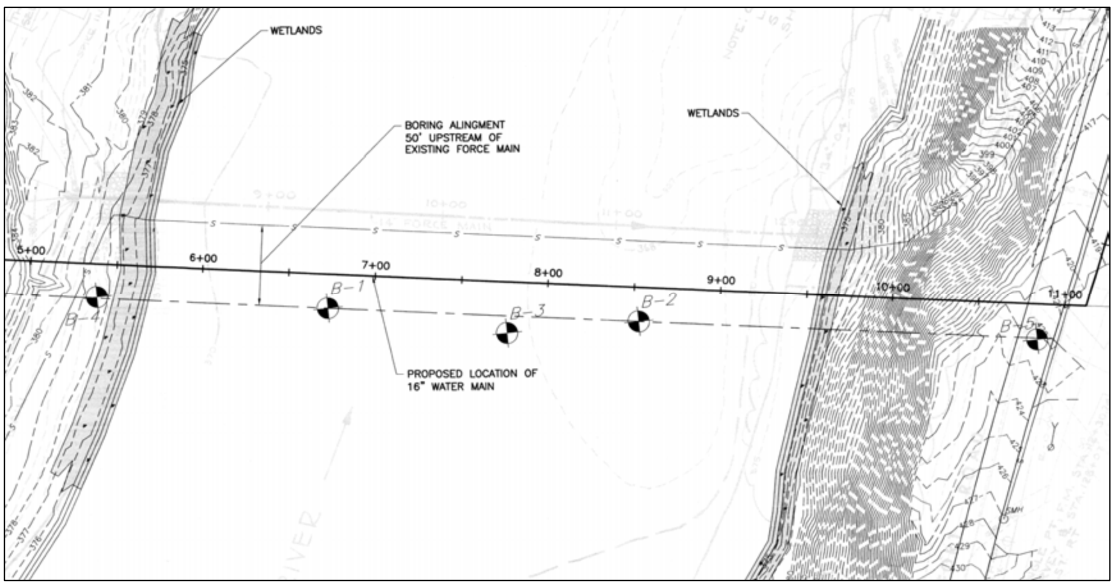

Figure 1: Aroostook River Crossing Plan

Discussions with the HDD contractors indicated that the bore could be initiated on the west side at the high point on River Road and continuously bored to the east shore. This approach offered tremendous benefits over open cut installation down the steep embankment and connection closer to the waters edge. Unfortunately this was not to be the case.

Installation

The HDD contractor, Enterprise Trenchless Technologies, Incorporated of Lisbon Falls, Maine, arrived onsite and set up for the boring. They utilized a Ditch Witch JT 8020 Mach 1, rated for 80,000 lb pullback, to complete the bore.

Approximately 700-feet of HDPE pipe was fused on the west side of the river. An unused railroad ROW close to the river provided an ideal lay-down area for pipe fusing. The boring machine was set on the east side of River Road to allow enough curvature to maintain sufficient cover over the pipeline on the eastern side of the riverbed. Immediately after beginning the boring, bedrock was encountered at about 15-foot depth. This was unexpected, as test boring B-5; less than 25-feet away was drilled to a depth of 60-feet. The drilling machine was reset to the south for two additional attempts, and encountered bedrock on both tries. It was determined that the bedrock on this side of the river must steeply dip from east to west and that it would not be possible to HDD from this location as relocating the boring machine to the west would make the drilling profile to steep to allow for enough coverage of the pipeline under the eastern riverbed.

In a collaborative problem solving approach, the District, Wright-Pierce, the drilling contractor and the general contractor determined the best solution to the problem was to relocate the boring machine from the eastern side of the river to the western side and bore from west to east. The boring contractor immediately remobilized and relocated the boring machine to the west shore.

Redesigning the bore to bore to proceed west to east did present several problems, the greatest being that 700-feet of HDPE pipe was on the wrong side of the river. The general contractor contacted a local tow truck service with a long cable and the pipe was towed across the river to the eastern shore (see Photos 1 & 2)

Photo 1: Towing pipe across river looking east

Photo 2: Towing pipe across river looking west

The other problem was that the HDD contractor could not bore from the west shore to the top on the steep embankment on the east shore. This required that the general piping contractor excavate a receiving pit on the west shore just uphill of the delineated wetlands. The excavation was just inside the 50-foot buffer requested by MEDEP, however, MEDEP waived the NRPA permit requirements for this "emergency" change in the construction methods, as long as the change minimized any adverse effects on the river.

This revised alignment actually shortened the length of the HDD by about 100-feet. The question was, could the remaining length of pre-fused pipeline be utilized. The general contractor was able to trench up the access way adjacent to the existing sewer force main and install the remaining length of HDPE pipe. At the end of the fused HDPE the pipeline transitioned back to ductile iron. Electro-fusion couplings were used at both ends to fuse mechanical joint adapters to the HDPE pipe. This was done to minimize the size of the pits required to make the connections and the difficulty in accessing the pipeline ends with a fusion machine.

The HDD water main installation proceeded flawlessly. Photos 3 & 4 show the HDD contractor set up on the west shore. The District, MEDEP, and Wright-Pierce observed the installation and were extremely pleased with the final product and the minimal impacts to the Aroostook River.

Photo 3

Photo 4

The total construction time to install the HDD portion of the river crossing (approximately 600-feet) took about 4 days, including the first day spent unsuccessfully attempting the bore from the east side.

This is a small fraction of the time it would have taken to complete the same crossing by portable dams and open cut excavation. A virtually identical water main crossing project was previously completed a few miles upstream of this project for a different District. This particular client specified ductile iron for the river crossing pipe material and elected to have the pipe installed by open cut method. From installation of the portable dams to pulling out of the river, this crossing took about 6 weeks to complete. The contractor had to address issues with heavy rainstorms raising the river water level and overtopping the porta-dams and coarse riverbed gravels requiring significant dewatering within the dams. Photo 5 shows this open cut installation with porta-dams.

Photo 5: Aroostook River water main crossing, Presque Isle, Maine

Cost Benefits

Installing the crossing by HDD provided significant cost benefits for the District. The direct construction cost savings of the HDD crossing verses the open cut crossing was likely on the order of $50,000 to $100,000, based on the installed costs of the Caribou project and the adjacent District project.

In addition to the direct construction cost savings, the District realized a substantial savings in engineering fees that would have been required to perform all of the permitting requirements of open cut excavation in the designated *outstanding river segment*.

The District also realized cost savings keeping to the projected design and construction schedules. If the District had opted for open cut, the additional time to complete the permit process, and the likely stipulations that would have been permit required, would have pushed the active construction from one summer construction period to the next year's construction window. Inflation alone, for the construction materials, would have likely added another 5%-10% to the project construction cost for a delay to the next construction season.

Conclusions

The success of this project was due in large part to the cooperative efforts of the people involved with the project, from the District Personnel to the State Regulators to the competent construction contractors. Thanks go to all who participated in the project.

Many lessons were learned during the design and construction of the project;

- Foremost, horizontal direction drilling of the water main across the Aroostook River provided a cost effective, timely, and appropriate method for delivery of drinking water from the new pump station to the distribution system.
- HDD installation had virtually no adverse environmental impact on the Aroostook River.
- Early conversations with State Regulators set the tone for design tasks and preferred construction methods.
- HDD, by minimizing environmental impacts, also required the least amount of permitting efforts.
- Materials selection was very important for a successful project and to meet the long term needs of the District.
- Knowledgeable HDD contractors were instrumental in commenting on the design and constructability of the proposed project.

Lessons from the Failure of Two Reclaimed Water Lines in California

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Abstract

The Landfill Reclaimed Water Distribution System consists of two pump stations and two force mains. Pump Station No. 1 pumps into a 640 m (2100 ft) long 90 cm (36 in) diameter Force Main under a static head of 150 m (500 feet). Pump Station No. 2 pumps into a 1125 m (3700 ft) long 75 cm (30 in) diameter Force Main under a static head of 90 m (300 ft). Both pipes are Ductile Iron Pipes with TR Flex joints, wrapped in polyethylene sheets and were supplied by U.S. Pipe. The lines were tested under a static pressure of 2000 kPa (300 psi) after completion. Subsequently the pipelines were connected to the discharge headers and tested under a static pressure of 2000 kPa (300 psi), at which time significant movements were observed at the pump station piping at both Pump Station No. 1 and Pump Station No. 2. The authors were engaged by the Sanitation District to investigate movements observed on the pipe headers at both Pump Stations during the pressure tests. This investigation focused on the adequacy of the design of the entire system, and the installation procedures during construction of the project. The manufacturer's recommended installation procedures are also reviewed. This paper reports on the details of this investigation and how appropriate corrective actions were taken to stop further movements in the lines once the pipelines were put back into service.

Site Visit and Data Review

At the onset of the investigation, the senior author visited the site along with county engineers and inspected the damage on the header lines during the pressure tests. He walked the alignment, inspected all of the pump stations, and controls. He also had an opportunity to meet with the personnel from the contractor and sale staff from the pipe suppliers for the project at the job site. The senior author also went over most of the files on the project in the offices of the sewer district and requested needed evidence to continue with the investigation.

The following documents were reviewed by the authors on this project during the course of this investigation:

- Movements recorded.
- U.S. Pipe Company's Product Literature.
- The Correspondence on the project.
- Surge Calculations by Flow Science, Inc.
- Photographs of the Force Main.
- Pipe Manufacturer's Drawings.
- Geotechnical Reports by 2 different local firms
- Specifications of the project.
- Engineering Drawings of the project.

Geotechnical Information

The site is located in the Los Angeles County. The site is underlain by bedrock and has natural average slopes of 2.5 H:1.0 V, with a maximum of 1H:1V. Vegetation on the slopes consists of low annual grasses and scrub oak. Two previous site investigations were carried out to characterize the geotechnical properties at the site:

- The first firm conducted a preliminary geotechnical investigation. Two boreholes were advanced to depths of 20 and 40 m (65 and 130 feet). During the drilling no groundwater or caving of soil was observed. Based on laboratory tests on samples collected during the investigations the following parameters were reported for the soils at the site:
    - Dry density: 1600 – 1867 kg/m^3 (99.8 - 116.6 pcf)
    - in-situ moisture content: 11.1 - 18.3 %
    - Max. Dry density: 1795 – 1858 kg/m^3 (112 -116 pcf)
    - Optimum moisture content: 12 - 16 %
    - Internal friction angle: 32-38 degrees
    - Cohesion: 0 – 75 kPa (0 - 1.55 ksf)
    - Sand equivalency: 3 - 4.
- The second firm conducted a subsequent soils engineering investigation. Nine shallow boreholes were advanced to depths less than 5 m (15 ft) along the length of the layout of the pipelines. Based on laboratory tests on samples collected during the investigations the following parameters were reported for the soils at the site:
    - In situ dry density: 1653 - 2021 kg/m^3 (103.2 - 126.2 pcf)
    - Natural moisture content: 12.5 - 24.3 %
    - Liquid limit (for fines): 26 - 49 %
    - Plastic limit: 19 - 33 %
    - Sand equivalency: 2 - 25

The soil typically found at the site was classified as Colluvium, consisting of soft, moist, porous, medium brown clayey to sandy silts.

Other relevant information from the investigations is as follows:

- The site has a fairly uniform soil strata consisting of moist, hard, mottled , light yellow silt-stone at depths ranging from 1.5 to 4.5 m (5 to 15 ft). The dry density of this soil was around 1732 kg/m^3 (108.1 pcf). The water content of the soil was around 18 percent.
- In areas near station 19+50, the soil was found to be a loose, light gray weathered silt-stone with a dry density of around 1539 kg/m^3 (96.1 pcf). The water content of this soil was around 18 percent.
- In areas near station 31+00, a light green, moist, moderately compacted sand was found to emanate a gasoline odor. The dry density of this sand was around 1940 kg/m^3 (121.1 pcf). Its water content was around 9 percent.

Pipe Materials

Two types of pipe materials were used in the construction of the system. Steel pipes were used for the pump station piping, manifolds, and headers. Ductile Iron pipes wrapped in polyethylene sheets were essentially used for the two force mains which were buried in soil depths of about 0.9 1.1 m (3 to 3.5 ft). U.S. Pipe and Foundry Company of Birmingham, Alabama supplied the ductile iron pipes and the joints.

Joints

The joints in the pipelines were TR Flex joints or Tyton joints. According to the U.S Pipe manual for the TYTON Joint pipes, restrained push-on joints for pipe and fittings are guaranteed for a water working pressure of 1725 kPa (250 psi) for sizes 76 cm (30 in) through 162 cm (64 in). However, despite the above limitation on working pressures, the lines were tested under a static pressure of 2000 kPa (300 psi) after completion of installation. Further investigation of existing systems using this joint indicated that the highest pressure at which U.S. Pipe and Foundry has used this joint successfully in 75 and 90 cm (30 and 36 in) sizes is only 690 kPa (100 psi), far lower than those acting on the pipelines at this project.

Information from Manufacturers

US Pipe and Foundry have been engaging independent Testing Agencies in the past to pressure test and report on the pipe joints. However, these tests are done in ideal laboratory conditions. Their applicability to joints in the field is questionable. Actual tests are necessary to verify assumptions made in the modeling and analysis. Even the little testing pipe manufacturer had done to date was mainly to determine the pullout ultimate strengths of the TR flex joints in various sizes and never on the amount of axial movements experienced in the joints at various levels of pressure. Unfortunately, all of the test data sent to us by the pipe supplier has not been of much use in this investigation when the main thrust of this investigation is to stop the axial movements in the pipelines.

3D FEA

Caesar II, a three dimensional computer program that implements FEA was used for the structural analysis of the pipelines and the pump station piping. The system was modeled to include all the bends, expansion joints, bellows, valves, and flanges. Actual weights of the bends and other joints from the manufacturer's brochures have been used in the analysis.

Assumptions

The following assumptions were made to simplify the analysis:

- The soil behavior approximated by a bilinear stress-strain model.
- The pipe behavior is approximated by a linear elastic stress-strain model.
- Consideration of translational and rotational deformation due to the Bourdon Pressure Effect was included. The Bourdon effect accounts for the longitudinal forces on the pipe from high internal pressure. This effect also causes straight pipes to elongate, or displace along their axes, and causes curved pipes, or bends to elongate along the line connecting the far and near points of the bends
- The system broken was into two separate systems for ease of analysis
    - System 1 consisting of Buried Line 1 from Sta. 00+50 to Sta. 21+43.27 and including Pump Station No. 1 Piping
    - System 2 consisting of Buried Line 2 from Sta. 22+13.27 to Sta. 59+69.64 and including Pump Station No. 2 Piping.

Modeling

The models used in the analysis included buried lines, pumps stations, and controls.

- Soil
    - $\phi = 20^{o}$;
    - Unit Weight = 1922 kg/m^3 (120 pcf)
- Steel Pipe
    - Elastic Modulus = 193 GPa (28.0 X 10^6 psi)
    - ν = 0.3
    - Pipe density = 7750 kg/m^3 (0.28 pounds per cubic inch)
- Ductile Iron Pipe
    - Elastic Modulus = 165 GPa (24.0 X 10^6 psi)
    - ν = 0.28
    - Pipe density = 7750 kg/m^3 (0.28 pounds per cubic inch)

Methodology and Results

In the analysis the following issues as related to the design, installation, testing, and operation were addressed:

Pressure Testing of the Lines for Acceptance

This was simulated by modeling the lines as buried pipes, with the exposed flanged ends blinded. Pump Station piping was not connected to the lines. The soil friction was varied from 0.2 to 0.5 to assess the sensitivity of the soil friction on the buried lines because of the shallow soil cover of 0.9 to 1.1 m (3 to 3.5 ft). Internal static pressure of 2000 kPa (300 psi) similar to the actual acceptance test was applied in the simulated analysis. The impact of slacks on the lines during installation was not analyzed. However, Bourdon Pressure which accounts for lengthening of pipe due to internal pressure has been considered in the analysis. The models have been formulated to allow for analysis in the future to include slacks in the line once field tests are conducted and/or additional information from US Pipe is obtained.

The effect of soil friction on Pipe Displacements for Buried Line 1, and Buried Line 2 were studied. Similarly the effects of soil friction on Axial Force, Shear Force, Bending Moment and Torsional Moment were also analyzed. After reviewing the effects of soil friction and considering the shallow depth of the soil and possible loosening during pressure testing, a soil friction factor of 0.2 is considered more appropriate in the analysis of the system.

System Pressure Tests

This simulated the actual pressure tests. This included connection of the line to the pump station piping, water tanks and the surge tanks. An internal pressure of 2000 kPa (300 psi) was applied to the system. Two cases were analyzed, with soil friction of 0.5 and 0.2. Only the more relevant cases with soil friction = 0.2 which represent the field condition are presented.

For Buried Line 1 and Pump Station No. 1 Piping nodal displacements and rotations at the tested pressure of 2000 kPa (300 psi), and as well as for the system operation pressure of 1550 kPa (225 psi). for the Header and Manifolds were studied in detail along with Moment, Axial and Shear force for the Header.

For Buried Line 2 and Pump Station No. 2 Piping, the element Axial Force, Shear Force, Bending Moment, and Torsional Moment were studied along with Local Forces and Moments and the Global Forces and Moments.

System Operation

This was simulated using an internal pressure of 1550 kPa (225 psi).

For Buried Line 1 and Pump Station No. 1 Piping nodal displacements and rotations for the system operation pressure of 1550 kPa (225 psi) for the Header and Manifolds were analyzed with Axial and Shear force for the Header. The Moment along the Header for the operating pressure case and the element Axial Force, Shear Force, Bending Moment, Torsional Moment were also calculated along with Local Forces and Moments and the Global Forces and Moments.

For Buried Line 2 and Pump Station No. 2 Piping, the element Axial Force, Shear Force, Bending Moment, and Torsional Moment were calculated with Local Forces and Moments and the Global Forces and Moments.

Thrust Blocks and Expansion Joints

Thrust blocks reduce the displacements and rotations of the restrained section of the line by providing additional restraining capacity. On the other hand, expansion joints relieve the stress in the line by allowing additional displacements/and or rotations. Usually for structurally stiffer system expansion joints will relieve overstressing, while for less stiffer system thrust blocks are more suited.

The lower system consisting of Pump Station No. 1 and the buried line is less stiff when compared to the upper system consisting of Pump Station No. 2 and the line. Thrust blocks may thus be more effective for the lower system. But once the Pump Station No. 1 Piping is isolated it becomes a stiffer system, and an expansion joint between the Surge Tank and other piping will provide additional stress relief. For the upper system expansion joints are more appropriate.

The analysis included the provision of Thrust block at the following location for the lower system.

- System 1: Provide a Thrust Block at Sta. 0+55, to isolate the Pump Station No. 1 Piping from the Buried Line. Provide an Expansion Joint between the Surge Tank and the Eccentric Reducers on the header. Amount of movement to be tolerated was 15 cm (6 in)
    - Global forces
        - Fx – 350 kN (78.8 kips)
        - Fy – 10.7 kN (2.4 kips)
        - Fz – 40.5 kN (9.1 kips)
    - Global Moments
        - Mx – 12.7 kN-m (9.4 ft-kips).
        - My – 1171.2 kN-m (862.9 ft-kips)
        - Mz – 77.4 kN-m (57.0 ft-kips)
- System 2: Isolate Pump Station No. 2 Piping from the Buried Section by providing an expansion joint between the manifold for the future pump and the main line flow meter on the 75 cm (30 in) dia. header. The amount of movement to be tolerated by the expansion joint shall not be less than 15 cm (6 in). Typical 3D FEA results on the effects of the axial stiffness of the expansion joints on shear force, bending moment, torsional moment, and displacement along the pipeline alignment are shown in Figs. 1-4.

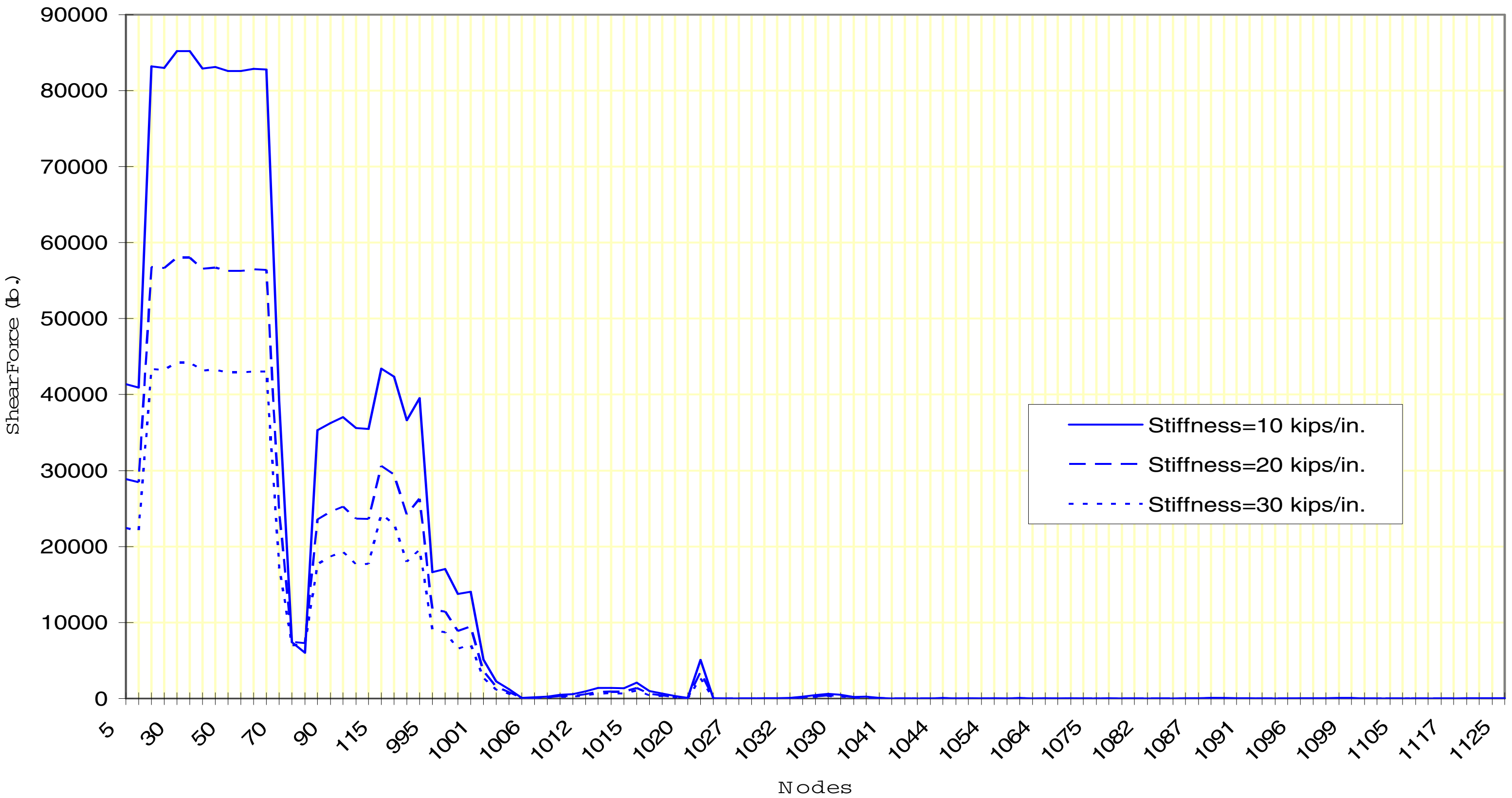

Figure 1. Effects of Axial Stiffness of the Expansion Joint on the Shear Force in the Pipeline

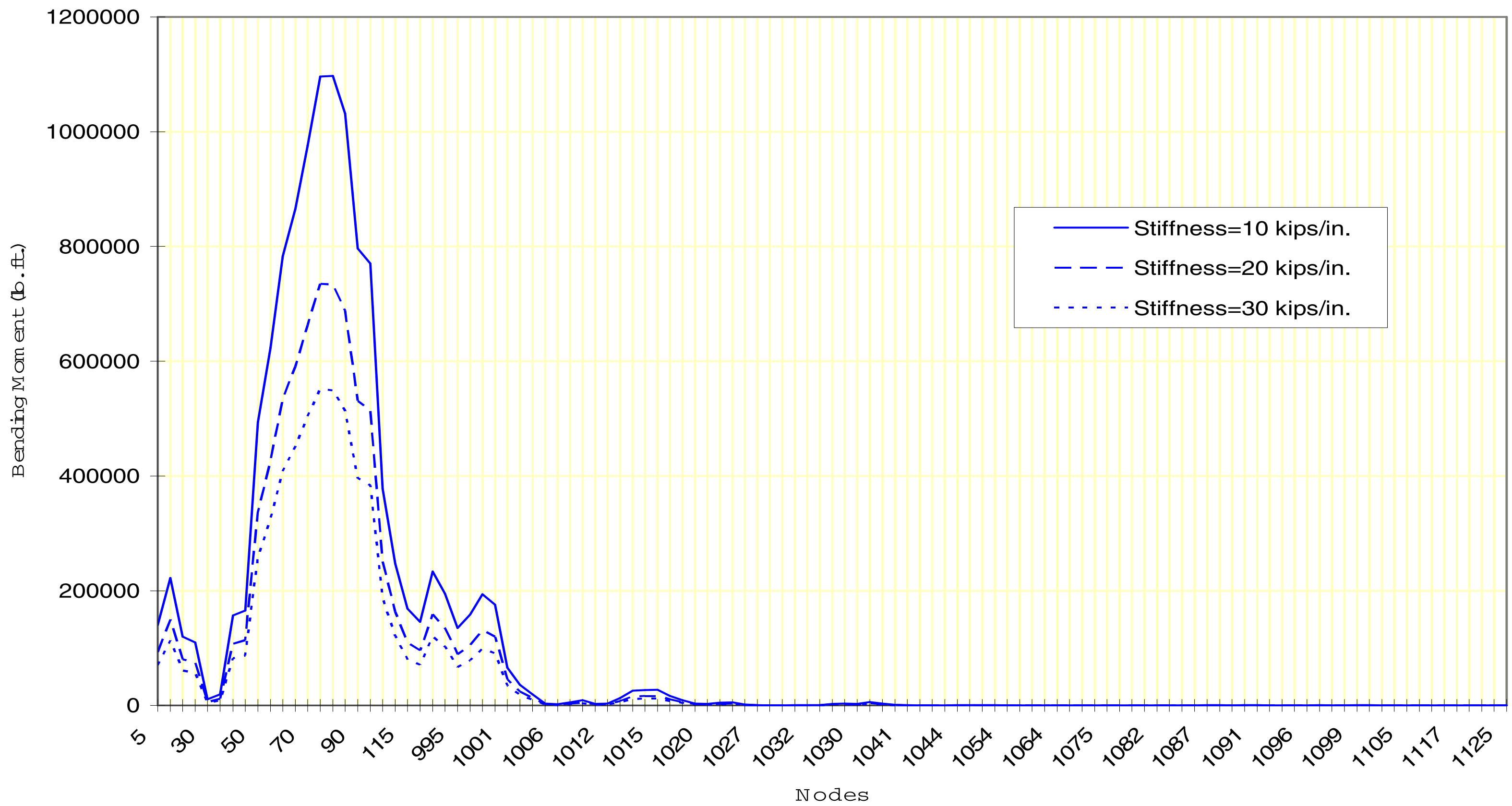

Figure 2. Effects of Axial Stiffness of the Expansion Joint on the Bending Moment in the Pipeline

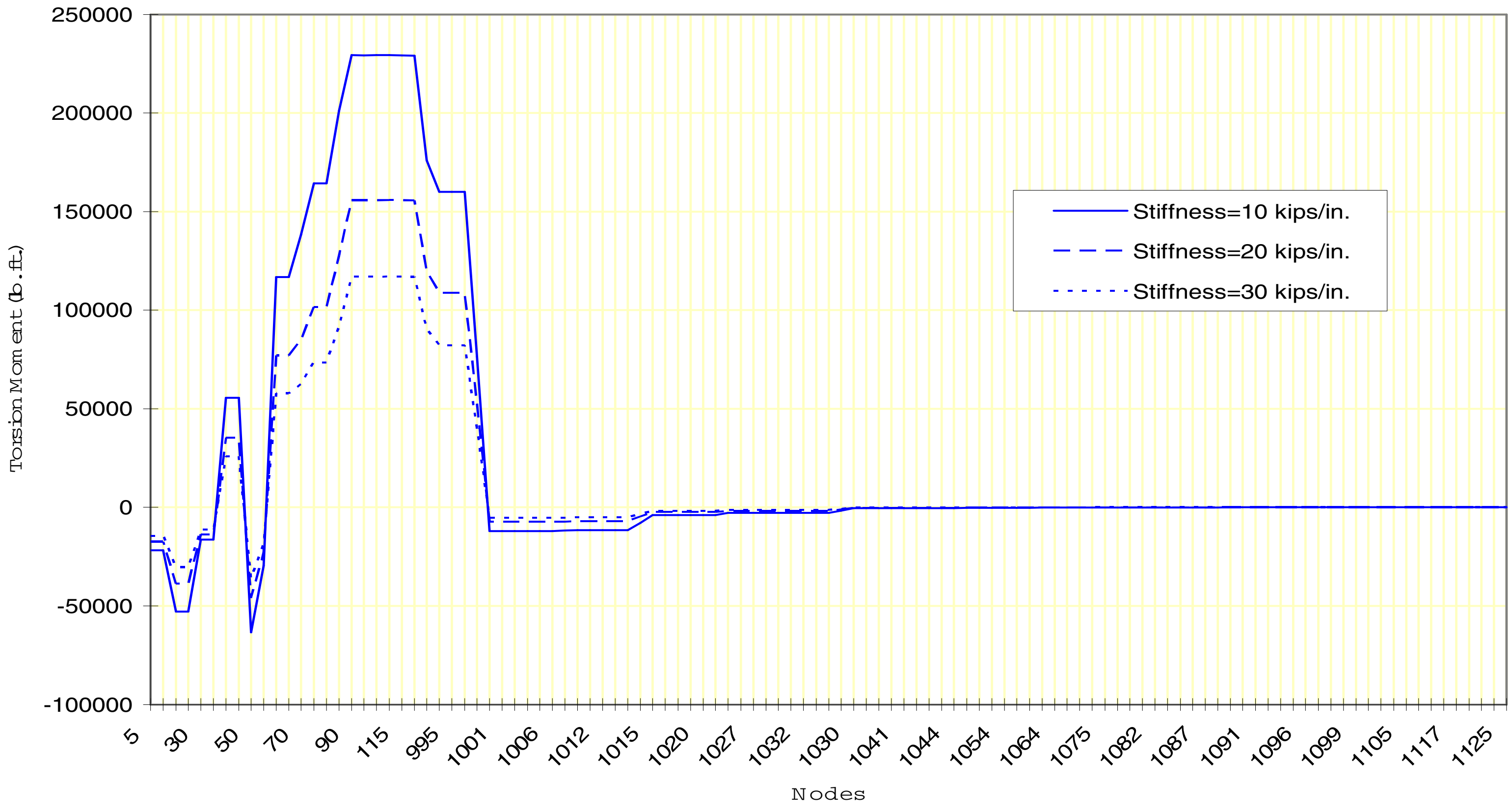

Figure 3. Effects of Axial Stiffness of the Expansion Joint on the Torsional Moment in the Pipeline

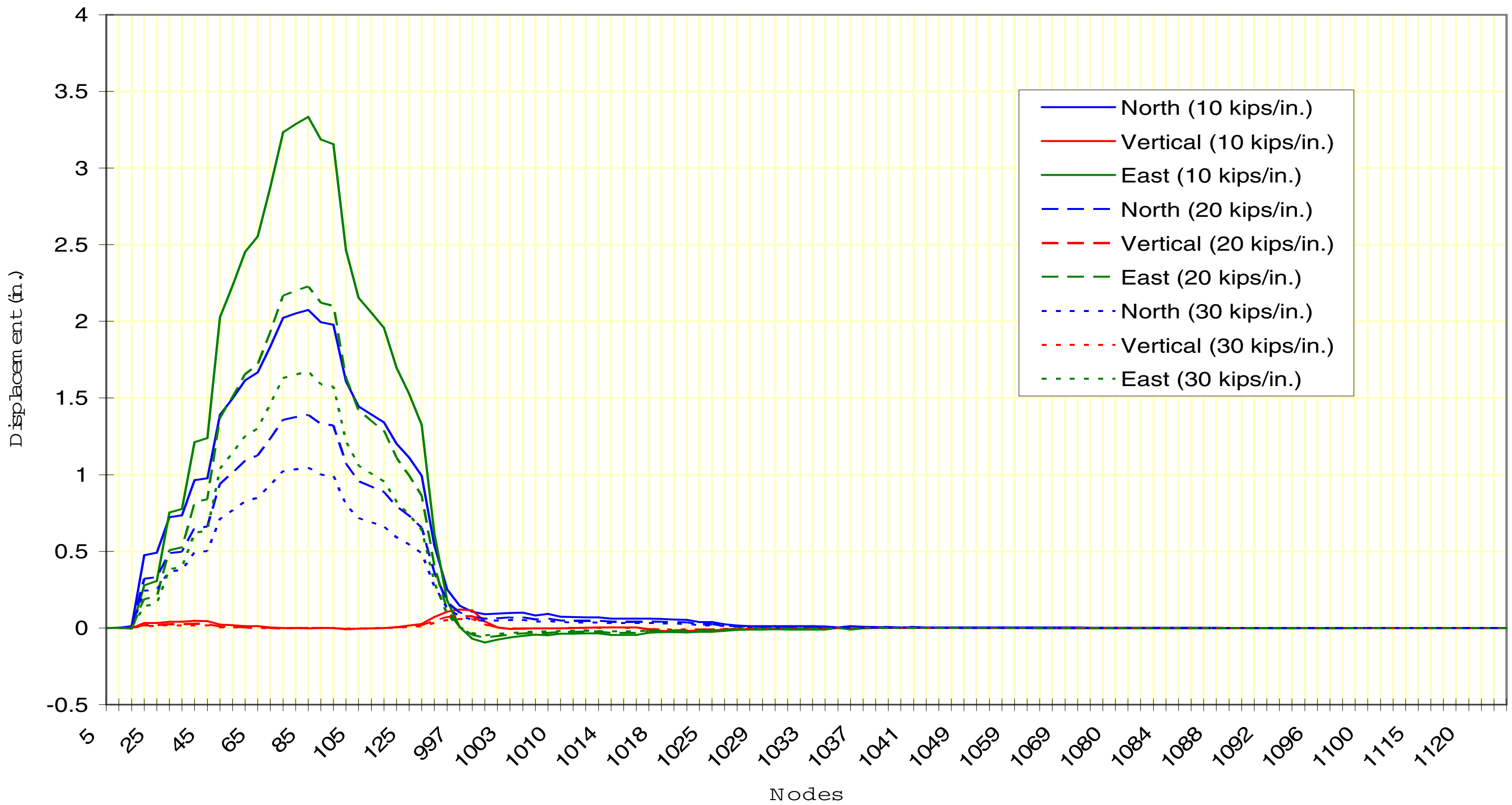

Figure 4. Effects of Axial Stiffness of the Expansion Joint on the Displacement in the Pipeline

Assessment of Current Condition of the System

In the foregoing analysis Bourdon Pressure Effects have been used for the elongation effect of pipes due to high internal pressure. This does partially account for the removal of the slack. However, this needs to be verified later once field data becomes available for modifying the analysis.

Causes of Movements and Recommendation

The following conclusions were reached from this investigation:

- In the selection, design, and implementation of the TR flex joints, one has to recognize that the tees involved in the joints have a pressure rating lower than even the joint itself. In the 30 and 36 inch size joints, the maximum rated capacity of pressure is only 1725 kPa (250 psi). If the joints are used in pressures higher than this, the factor of safety is compromised and one will see movements higher than normal. The rated capacity of the tee that locks the joint in restrained condition is even less, sometimes as low as only one-half of the capacity of the overall pipe. The axial movement performance of this joint has never been studied by the pipe supplier and this poses a major problem in that this could be somewhat of an experimental joint when used in high pressure range in larger pipe sizes. Also, no other force mains in this size range with TR flex joints are in the ground to provide us a proven track of satisfactory service of this joint in pressures higher than 690 kPa (100 psi). It is entirely possible that the pipe manufacturer's sale staff sold the wrong joint for this project by their overzealous marketing of a new joint.
- In steep slopes of ground, it is always recommended that the direction of laying be down slope and if the contractor had followed a laying sequence other than this, the contractor's field crew would not have been able to remove all of the slacks in the joints to extend the pipe length to its fully extended positions during laying. And any slacks left in the joints would stretch while the line is put under pressure and would rebound due to ground pressure and due to the elastic rebound in the material used for tees, when the internal pressure is lowered.
- The geometry of the loops in the alignment of the pipeline also would give rise to some movement in the axial direction whenever the pipe is out under pressure. However, the order of magnitude of this movement would not be substantial compared to those resulting from the above two factors.
- Appropriate specifications for the thrust blocks/expansion joints for the lower system and for the expansion joints for the upper system were given to the owner. These corrective actions were implemented to prevent further movements in the pipelines.

Risk Management Begins in the Planning Stage

By Will Worthington, PE, [1] and Robert Homer, [2]

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Abstract

Many of the most important measures to reduce pipeline risk management are made in the very early stages of formulating a pipeline plan. These decisions, if tempered by experience and judgment, will greatly facilitate the many actions that will follow to design, construct and operate a pipeline. Conversely, if knowledgable managers do not make these decisions and actions in a timely manner, the path to efficient operation and effective risk management will be an uphill struggle. The successful implementation of a pipeline facility plan entails a myriad of actions and decisions, beginning in the conceptual stage and continuing into the operation and maintenance stage. The purpose of this discussion is not to review the step-by-step procedures for pipeline planning, design, construction and operation; it is rather to highlight some of those early considerations and decisions that we have learned from experience are very important but often overlooked. Discussion will include these decisions that will be made in the planning and design stages of a pipeline project's evolution:

- Early identification of pipeline project need
- Identifying high consequence areas
- Planning for outages
- Planning for condition assessment

Introduction

As we know, the general public takes pipelines for granted. Water flows from our faucets, our toilets flush, the gas pumps deliver the fuel for our cars . . . and people by and large to not know or care too much about where these fluids come from or where they go. We submit that this may be indeed a compliment and resounding endorsement by the public for the work of practitioners in the field. By some standards, the underground pipeline is the consummate engineering achievement in its functionality and elegant manner of fulfilling vital needs. As we also know, the successful implementation of a pipeline facility plan entails a series of important actions and decisions, beginning in the conceptual stage and continuing into the operation and maintenance stage. Many of the early decisions will ultimately have a great impact on the safe and reliable operation of the pipeline. The purpose of this discussion is not to review the step-by-step procedures for pipeline planning, design, construction and operation; it is rather to highlight some of those early considerations and decisions that we have learned from experience are very important but often overlooked.

The earliest of these decisions may well be the timely recognition of the need to build a pipeline. The decision-maker must follow with proper program management, budgeting, design, construction and operation. Reviewing the performance of a few successful, as well as some not-so-successful pipeline projects will be instructive in highlighting a few key steps along the way.

Figure 1 – Petroleum pipeline booster station, Colorado. The petroleum pipeline industry is regulated in planning for risk management (Pipetech International photo)

Beginning with a Vision

Our underground pipelines are a marvelous, elegant, and expensive infrastructure investment. Each of these facilities is born as a vision in someone’s mind and grows to reality. There is a need to be met, or a purpose unfulfilled, and an idea is born. Organizations must make room somewhere within their framework for long range planning to be done . . . for there to be people with the assigned task of planning ahead for those pipeline projects to fulfill our needs. These needs may be a new facility or major rehabilitation in order to:

- Accommodate growth
- Replace aging infrastructure
- Tap a new petroleum or water source
- Provide redundancy
- Eliminate a dangerous condition
- Address a political requirement

As with so many tasks, this planning is not the work of an individual, but rather the work of a team of people operating under the direction of effective management that has established an atmosphere where comprehensive planning can be done. Appropriate communication among various departments is a key element in this plan formulation arena. As an obvious example, a decision by a city planning department to allow rezoning to higher densities should also address the needs for expanded water and sewer service. A not-so-obvious example might be the gradual realization by the public works department that the maintenance costs of a pipeline are increasing every year and the reliability is decreasing, and it will be cost-effective to replace this pipeline.

Failure to recognize, or ignoring a growing need for a pipeline sounds like a perfect way of gradually turning up the heat on a pipeline problem. As the problem gradually reaches the boiling point, the responsible management may face a steep, uphill struggle to provide a pipeline to meet the need. Inadequate budget, an unrealistically tight schedule, shortcuts in design, and inadequate staffing are likely to lead to serious problems during the life of the project. Conversely, the team that recognizes the need for a new facility in ample time, and lays the groundwork for its implementation will make the path towards implementation much easier.

Identifying High-Consequence Areas

Water and wastewater pipelines are often installed in locations that, if they fail, pose a significant risk to the surrounding community and environment. Often labeled as "high consequence areas", these are areas where pipeline incidents or failures would likely severely affect public safety, create service disruptions, and cause significant damage to property and the environment. Owners must assess pipelines in these areas for safety risk, and give them special attention. This process will be of greatest value if it is performed during the planning for a new facility, when the option of risk management by modifying the pipeline alignment is still available. If the pipeline already exists, the risk assessment is no less important, although the risk management options are more limited.

This risk assessment process should qualitatively evaluate the "high consequence areas," and should be used to direct the appropriate response to the risk created by a potential failure. The areas where the highest consequence of rupture is identified should be candidates for a systematic and rigorous program of condition assessment to identify any tendency for deterioration or unusual loading conditions. If a hazardous condition is identified, the owner should be prepared to repair, replace, or remove the pipeline form service, if the pipeline is found to be defective.

"High consequence" will be seen to be a function of many different variables, the most obvious of which is "high consequence" in the spatial sense, such as densely developed or densely populated urban areas. This is perhaps the easiest to recognize and the avoidance options are readily apparent. In some instances the option to avoid the area may become apparent during the early planning stage . . . to route a new pipeline in another direction. Petroleum pipeline planners enjoy an advantage over water/wastewater planners here. Water and wastewater planners must provide the water transmission mains or the sewer lines expressly to serve these densely populated areas. A "High consequence" category may be the result of other variables, such as:

- Geologic hazards, such as areas of earthquakes, faults, subsidence, or fissuring
- Economic factors, such as unusually high valued real estate or dense utility corridors
- Land uses, such as school grounds, shopping malls and hospitals
- For projects in the planning stage, there may be some areas of limited jurisdiction and lack of eminent domain authority, e.g. Indian reservations, railroad rights of way, and other federally and state protected areas
- Certain areas are "high consequence" for environmental and public health reasons, such as drinking water supplies
- Aesthetically sensitive areas, such as monuments and natural areas
- Certain areas are "high consequence" for social reasons, such as Indian reservations, cemeteries, and historic sites

Risk assessment of high consequence areas involves understanding the hazards along the pipeline route, the overall condition of the pipeline itself, as well as the overall consequences if a failure were to occur in a specific area. In its most simplest form risk can be illustrated as:

Risk / loss = Failure magnitude x Failure Frequency x Consequences

To complete a risk analysis for a pipeline in a high consequence area, the potential failure characteristics for the pipeline segment need to be identified and these characteristics of failure need to be quantified. This can be done by using a risk matrix or risk indexing system. The qualitative risk models are unique, since the scoring derived is indexed to specific failure frequency and outcome or probability ranges. This allows for a more clear determination of which pipe segments are of high consequence concern.

A risk matrix helps determine where attention should be focused relative to other areas of lesser consequence, but also identifies the failure characteristic that most significantly contributes to the overall high consequence risk of failure. A simple risk matrix for high consequence areas can be illustrated like this:

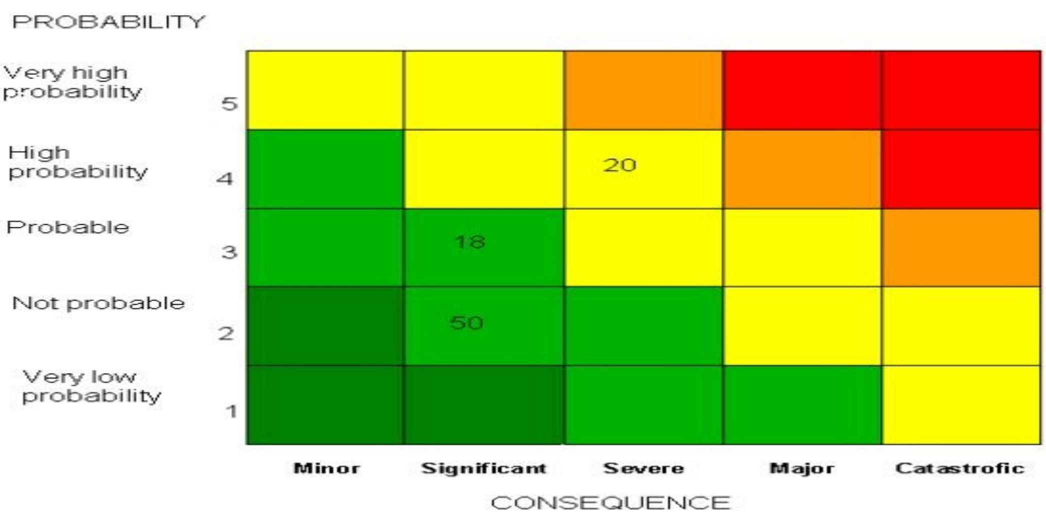

Figure 2 – Risk Matrix

Benefits of utilizing a risk matrix for assessment of high consequence areas:

•Identification of primary risk causation factors

•Relative risk ranking of all pipeline sections

•Prioritization of replacement or rehabilitation planning

•Determines areas of missing or incomplete information on pipeline segment

•Recognition of high-risk pipeline segments for more detailed evaluation.

Quantitative risk ratings and definitions utilizing the risk matrix

Risk Rating	Definition
Very High Probability / Catastrophic loss	Primary high consequence risks requiring immediate attention due to potential catastrophic failure consequences. Risk reduction or mitigation planning needs to be developed. Pipeline owner contingency plans required, and response plan in place in event of pipeline failure. Risk monitored and updated regularly.
High Probability / Major consequence	Significantly serious to warrant attention slightly below Very High Probability. Should plan on developing risk reduction or risk mitigation strategies. Owner or operator of the pipeline should have contingency plans in place for pipeline failure. Risk monitored regularly.
Probable / Severe consequence	Lesser probability but may cause distress or inconvenience for short term. Owner or operator may consider using a third party emergency response or planning for a failure Risk monitored
Very low / not probable / minor consequence	Risk of failure managed using regular or non-specific emergency planning arrangements. Minimal monitoring and oversight unless risk determination or conditions change to elevate to a higher rating.

Benefits of Quantitative Risk Assessment for use in high consequence areas

• Specifically focused replacement or rehabilitation programs

• Confirms appropriate actions taken on high consequence areas.

• Demonstrates due diligence in pipeline maintenance

• Definable Benefits for pipeline replacement or rehabilitation

• Assists in correct application of resources and money

• Can be utilized for “what-if scenarios”? Evaluation studies

• Provides for cost / benefit analysis

• Risk assessments are invaluable as they provide a snapshot of the vitality of a pipeline system and the degree to which various potential failures can occur using a consistent, repeatable methodology for every segment of pipeline. This assists in the reduction of the possibility of bias and error and ensures that no part of a pipeline is left unexamined.

Knowing the potential failure, and consequence, of a pipeline distribution system allows for the selection of the most appropriate mitigation measures to maximize the overall operation of the pipeline system.

Certainly there is no simple recipe to avoiding or mitigating damage in high consequence areas, but the first step is to identify these areas as early as practical. This holds true whether we are planning a new project, or managing the risks associated with existing project. Good portions of experience, ingenuity and creativity will minimize the impact of high consequence areas, and greatly enhance a risk management program.

Planning for Outages

We have learned that it is not realistic to assume that pipelines can operate 24 hours a day, 7 day a week, indefinitely. This realization is something of a paradigm shift in the last few decades for those of us in the water/wastewater pipeline business, brought about by several factors. For one, the older pipelines began to show their age in the form of deterioration, leakages, and complete failures. For another, twenty five years ago there just were not many techniques to inspect or repair pipelines even if they were empty, but this has changed. For example, one of the most effective means of testing the structural integrity of pipelines constructed of Prestressed Concrete Cylinder Pipe, or PCCP, is the electromagnetic test that was first used in 1997. This test requires that the pipeline be taken out of service to allow personnel and equipment pass through the dewatered pipeline. Likewise, many pipeline repair and rehabilitation techniques have been developed that will greatly extend the useful life of a pipeline, but require that the pipeline be empty or depressurized for installation.
(Note to reviewers: some photos are removed to facilitate email transmittal)

Figure _ - Post-tensioning a PCCP pipeline to restore structural integrity, a technique first used in 1991. Standard practice is to depressurize the pipeline during installation.

Although some pipeline facilities can operate for long periods of time, there will eventually be a need to take the pipeline out of service for inspection, testing, and repair. We must anticipate that there will be outages, and we can either be proactive or reactive. Stated differently, we can either have planned outages or unplanned outages.

Planned outages can be accomplished in several ways.

- Parallel pipelines - Parallel pipelines to provide peak capacity may allow one of the lines to be taken out of service for inspection and repair during off-peak periods.
- Regulatory storage - A storage reservoir can be provided to provide service for a pre-determined period, allowing upstream pipelines to be taken out of service.
- Temporary bypass – The state of the art allows a temporary bypass of virtually any pipeline facility . . . but this can be quite expensive. It may be cost-effective in comparison to other options, but a disadvantage to this option is that it requires extensive planning and preparation time.

Figure 3 – In this planned outage a temporary bypass of a sewer main allows installation of cured-in-place lining
(Pipetech International Photo)

The point to be made is that the decisions to provide for planned outages will be made early in the plan formulation stage, and wise decisions in this regard will go a long way in facilitating inspection and repair of the pipeline. Risk management goes hand-in-glove with a comprehensive inspection and maintenance program.

Planning for Condition Assessment

Condition assessment has made quantum leaps in the past few decades, and the pipeline planner is well advised to anticipate the use of the latest technologies for this purpose. For instance, it has become standard practice to conduct condition assessment of PCCP pipelines using two recently developed technologies: acoustic emission and electro-magnetic. These technologies were introduced to the industry nearly simultaneously in 1997, and each has certain requirements for the testing. If the pipeline is designed and constructed with these needs in mind, the inspection and testing of the pipeline will be greatly facilitated.

The eddy current testing, for example, requires that people and equipment enter the pipeline to perform the test. The rate of testing is several miles per day, and for reasons of personnel safety and efficiency, there should be access manholes about every ½ mile. Unfortunately some pipelines lack the characteristics to facilitate the electromagnetic test, and in some rare cases the test is virtually impossible to conduct for operational reasons.

Figure 4 –Magnetic testing introduced in 1997. Manned equipment requires safe pipeline access (Pure Technologies photo)

In some instances, the distances between access manholes will be many miles. In other instances, the distances between in-line valves are so great and the condition of the blowoff valves is so poor, that dewatering the pipeline is a major operation in itself. This test would be greatly facilitated (and risk management greatly improved) if the design requirements included access manhole spacing not greater than ½ mile, and in-line control valve spacing not greater than every 3 miles.

Another very effective test method for PCCP condition assessment is the acoustic emission testing that is now widely used. This test method enjoys much of its popularity because it is conducted while the pipeline is in operation, however it still requires some preparation for the test to be effective. The test requires the insertion of hydrophones into the pipeline and the spacing of the hydrophones is critical to the effectiveness of the acoustic test. For larger pipelines, spacing of 2000 feet is adequate, while smaller diameters will require spacing not greater than 500 feet. Some existing pipelines have little or no available insertion points within the required spacing, and the performance of the acoustic test becomes quite difficult. Depending on the vendor’s methods, the insertion valve will need to be up to 2” full port diameter. This test would be greatly facilitated (and risk management greatly improved) if the design requirements included access manhole spacing not greater than ½ mile; with a 2” ball valve in each manhole, and positioned to facilitate hydrophone insertion.

Figure 5 - Acoustic Emission Testing, introduced in 1996 requires insertion of hyrophones at regular intervals
(Pressure Pipe Inspection Company photo)

We also suggest that certain condition assessment during the initial acceptance testing has merit and will enhance a risk management program. Petroleum pipelines have employed rigorous weld testing as a project is constructed, and the water pipeline industry would be well advised to follow suit. For example, leak detection has made great improvements in recent years and presents a new tool that might be of great benefit immediately upon completion of construction. The magnetic testing of PCCP upon completion could detect any anomalies prior to acceptance by the owner. Detection of hydraulic transients has also made great strides, and a hydrodynamic pressure test to compliment the traditional hydrostatic pressure test is now feasible. By conducting these tests before the pipeline goes into operation may identify problems that need immediate attention, and will provide a baseline for comparison of tests during the life of the pipeline.

Different pipeline materials and different liquids will employ different test methods, but the point to be made is that these methods and their requirements should be identified and addressed early in the process.

Conclusion

Risk management has evolved into a highly specialized field, yet it is perhaps an attitude as much as a concept. In a certain respect, an effective risk management program is similar to an effective safety program on a large construction project. Those projects where safety is accorded a high priority by management seem to have a low accident rate . . . and some might regard this as good luck. Yet experience tells us that we make our own luck when it comes to safety, and the same can be said of risk management.

The up-front work that is needed for successful risk management begins with a program to identify the needs for pipeline projects as far in advance as possible to permit the orderly implementation of the project. The early tasks in the process of formulating the pipeline plan will include the areas we have touched upon, namely careful attention to high consequence areas, planning for pipeline outages, and planning for condition assessment.

Risk Management and Pipeline Safety Committee Presentation

"Inline Assessment of Transmission Pipelines
In The Oil and Gas and Water Sectors"

Brian Mergelas,[1] PhD
Prof. David Atherton,[2] P.E.
Paul Passaro,[3] P.E.
and
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Introduction

Pipelines are pressure vessels used to transport products from one location to another and as such their operation falls under the US Department of Transportation. In principle, the Office of Pipeline Safety (OPS) has oversight on all transmission pipelines; however, in practice, regulations only exist for oil and gas pipelines. Spurred on by several spectacular and deadly pipeline failures starting in the 30's, the industry is now fully regulated. Present regulations include both risk management as well as mandatory condition assessment in high-risk areas. There are presently no regulations pertaining to risk management, pipeline assessment or pipeline safety in the water sector. It is not clear whether the fact that water transmission pipelines seldom cross state boundaries or the perception of lower consequence failures for water mains means that the water sector shall remain largely unregulated. It is clear that just as there is a strong desire to operate petrochemical pipelines up to and beyond their design life, there is a similar drive for water pipelines. The replacement value of water pipelines in the US is estimated at over $700,000,000,000. Comparing and contrasting approaches to condition assessment and risk management in these two sectors will certainly strengthen the emerging best practices.

Consequence of Failure

Potential hazards of petrochemical pipeline failures include environmental contamination as well as the possible explosions. Failures have lead to substantial property damage and have often tragically lead to deaths. Apart from the negative publicity that an operator receives, financial impact to a pipeline operator may range

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from lost revenues, substantial fines to court awarded damages that dwarf the cost of any property damage.

Even when a large water transmission pipeline fails, the negative publicity for a water utility is often short lived. They are very few deaths associated with water pipeline failures even when the lines have failed catastrophically. In populated areas, the water can cause significant property damage. These damages are often covered by individual property and business insurance policies, however in some circumstances utilities choose to assist those who are affected. While lost revenues associated with a failure are typically small in residential areas, there may be significant economic impact if the pipeline supplies an industrial user such as a textile mill, brewery or other manufacturer.

In-Line Condition Assessment Technologies

There are a variety of inspection technologies that are commonly used to perform inline inspection of steel transmission pipelines in the oil and gas sector. The choice of technology will often depend on whether the product being transported is in liquid or gaseous state and the suspected type of damage in the pipeline. In general, the industry is mostly concerned with localized corrosion, stress corrosion cracking and mechanical damage.

Corrosion pitting may be detected using magnetic flux leakage (MFL). The technology is based on using permanent magnets to create a magnetic circuit between an inspection tool or pig and the pipe wall. Magnetic flux will leak out of the pipe wall if there is any sort of defect present. Sensors close to the pipe wall can detect this leakage flux and detailed analysis can be used to estimate the size of localized corrosion. The technology is heavily influenced by inspection conditions such as tool and sensor lift off, tool speed, line pressures, corrosion pits, welds, magnetic hard spots and in general, there is reduced sensitivity to external defects and poor sensitivity to cracks.

In liquid filled pipelines, the use of Ultrasonic Technology (UT) tools is common. These tools send ultrasonic signals at the pipe wall and measure the time of flight as the signal bounces off the near and far pipe walls thus giving a measure of pipe wall thickness. The liquid helps to couple the sound from the inspection tool (or pig) to the pipe wall. While this technology may be deployed in a gas filled line, this requires running the inspection with a "liquid plug" surrounding the pig and complications in removing all of the liquid after an inspection run sometimes arise. By changing the angles of the UT transmitters, it is possible to look for cracks of various orientations as well and some specialized crack detection tools are commercially available. These tools are particularly valuable when there is concern over the presence of Stress Corrosion Cracking (SCC) in the pipeline. A variation of such crack detection tools uses EMATs (Electro-Magnetic Acoustic Transducers) that cause acoustic waves to propagate in the steel pipe wall by creating a strong electromagnetic pulse.

Prior to running the first inspection of a pipeline using one of the technologies mentioned above, it is common to run a cleaning pig through the line followed by a caliper pig. Caliper tools give a measure of the inner diameter and bends of a pipeline and are useful for detecting installation damage or other internal restrictions that might otherwise make a pipeline unpiggable.

It is also possible to use Remote Field Technology (RFT) to detect defects in the steel pipe wall. These tools use an electromagnetic transmitter and receiver which are indirectly coupled once out through the pipe wall, along the outside of the pipe and back in though the pipe wall at a remote spacing. The double through wall transmission of the signal allows for estimates of the pipe wall thickness (and other properties). Sensitivity to SCC has been successfully demonstrated in laboratory conditions, however inspection rates are slow by comparison to other inspection systems and extensive velocity control is required in practice. RFT does offer the advantage that the sensor on board the tool does not need to be in direct contact with the pipe and while it is best to have the detectors close to the pipe wall, this is not as important for RFT as it is for MFL testing. Generally speaking, the resolution of present day MFL tools tends to be higher than RFT tools.

In the water sector, there are many different types of pipe in addition to the steel found in the oil and gas sector. Operating pressures also tend to be significantly less and with the exception of PCCP, most materials will tend to leak before they fail. Most of the technologies used for the oil and gas sectors have limited application in the water sector. As discussed below, the access to water pipelines is typically poor by comparison to access to petrochemical pipelines. Generally, MFL and UT tools have large diameters (in order to accommodate the internal electronics required to acquire and store large quantities of data). MFL performs poorly when internal cement lining cause a lift off between the tool and the pipe wall; UT tools are adversely affected by internal tubercles. RFT has been used commercially in iron mains up to 12” in diameter using fire hydrants as access to the lines. Because the system is not as sensitive to tool lift off, smaller access, and internal lining do not create insurmountable obstacles.

Sahara® leak detection has been used in a host of materials to find small leaks in large diameter mains (>12”). The system works by introducing a sensitive acoustic device into a pipeline through a 2” tap. As the cabled probe passes a leak, the signal is reported in real time and the precise location of the leak may be pinpointed on the surface. Once the inspection is completed, the entire assembly is retracted through the initial entry point.

For Prestressed Concrete Cylinder Pipe, it is defects in the structural prestressing winding that are of concern. In the 1990’s Professor David Atherton of Queen’s University developed an enhancement to the RFT technology called Remote Field Eddy Current/ Transformer Coupling (RFEC/TC). In this system, the remote field that is sent from the transmitter to the receiver is amplified by the external prestressing winding through a transformer coupling effect. Breaks in the windings may be detected and quantitative estimates made. The system is also influenced by all electromagnetic properties of the pipe and surrounding area.

Deployment of Technologies

Transmission pipelines in the oil and gas sector are now designed with future inline inspection in mind. As such, launchers and receivers that provide full bore access to the pipeline are incorporated. Butterfly valves are not used as mainline valves as these would not allow an inspection pig to travel. Up to 800 miles may be inspected during a single inspection run. All data is collected and stored on board the inspection vehicle for future analysis. Since the movement of product is a major economic driver, with owners not being able to recuperate lost throughput until the end of the life of a given well, there is a preference to run the inspection tools as quickly as possible through the lines. In order to ensure acceptable levels of data quality, some advanced tools employ active velocity control systems that allow for product by pass.

It is important to note that in the past not all transmission pipelines were designed to be inspected. In the USA, it is estimated that a third of the high pressure transmission pipelines are considered unpiggable because of access and valve restrictions, multi-diameter designs, impassable fittings and a myriad of other configuration issues. Notwithstanding access issues, regulations in the oil and gas sector now require all pipelines to be inspected on a continuing basis. These regulations have meant some pipeline operators have spent considerable amounts of money "upgrading their pipelines". The inspection requirements have also encouraged the pipeline inspection companies to invest significant amounts of money creating specialized tools to overcome some of these pipeline obstacles.

Most water transmission pipelines would be considered unpiggable. It is common to use butterfly valves throughout the system and often mainline valves are undersized by comparison to the pipeline diameter. These internal bore restrictions have little impact on the hydraulic performance of a pipeline and can reduce the cost of a new installation, however the costs associated with future inspections may be high. It is uncommon for any sort of launcher or receiver to be included in a new design, and any such setup is normally designed with a cleaning pig in mind as inspection pigs are not common in the water sector. In most cases the access to large diameter pipelines (36"- 144") is through man holes (18"-36") or through smaller diameter flow meter points (~2").

There has been some work with tethered inspection systems using RFT. These tools can be deployed into metallic mains up to 12" in diameter through a fire hydrant and may travel several 1000 ft at a time. In larger diameter pipe, the Sahara® sonic leak detection system may be deployed though 2" taps and may travel as far as 6000 ft in a given inspection. RFEC/TC systems are typically deployed after dewatering a pipeline. These systems are "walked" or "driven" through a pipeline and while there is no limit to the total distance to be inspected, inspection rates of up to 7 miles a day are common. In the unregulated environment, the water industry as a whole is slow to make large investments in upgrading access to its infrastructure;

however individual owners are working closely with inspection companies to optimize expenditures between pipeline modifications and advances in the use of unmanned robotic systems. Many proactive design engineers are also promoting the inclusion of 2" taps strategically placed along new installations to facilitate future Sahara® inspections.

Failure Analysis & Repairs

ASME B31G is a standard that is used by the oil and gas sector to determine the remaining strength of corroded pipelines. It is based on the size of a corrosion pit and can be used to calculate the maximum allowable operating pressure (MAOP) for each corroded area of a pipeline. The MAOP calculation is modified using R-Streng when looking at non-localized corrosion. Both B31-G and R-Streng are based on calculated and experimental results relating the physical condition of the steel pipe wall to the burst pressure of the pipe. Detailed local WT information is required.

Operators generally have two choices in operating their pipeline. They can choose to adjust the operating pressures so that they do not exceed the MAOP. Most often they will also repair the worst defects (those with the lowest MAOP). Replacement of large sections of pipe is difficult and repairs usually consist of cutouts, welding external plates or external strengthening.

In the water sector, detailed condition assessment information is generally only available for PCCP. Using RFTC, it is possible to estimate the number of broken prestressing wires for each pipe along the length of a PCCP pipeline. Detailed structural analysis of PCCP has been conducted by Simpson Gumpertz Heger based on their previous limit state work when writing the AWWA C304 design standards for PCCP. For a pipe of a given pressure and known operating condition, and given the estimated number of wire breaks, it is possible to prioritize pipes for rehabilitation based on the probability of failure.

Rehabilitation options for PCCP include the replacement or strengthening of distressed pipe sections. Replacement may include physically removing damaged sections or using a structural liner inside the pipe. PCCP may be strengthened from the outside using post tensioning strands. Depending on the design assumptions, carbon fiber may be used to strengthen a pipe or as a stand alone structural liner.

While it is possible to use a modified version of B31G and R-Streng for metallic water transmission pipelines, localized WT information is difficult to come by. General loss of WT over a large area or sampling of WT information along the length of a line is sometimes used as an indication of when to rehabilitate or replace a pipeline section. In general, rehabilitation of metallic mains is performed through slip lining the existing main although recently carbon fiber has been used to repair some larger mains.

Risk Management

Pipeline Risk Management is regulated by the Office of Pipeline Safety under the Department of Transportation. While in principle the OPS are responsible for all transmission pipelines (including water pipelines), in practice they are only concerned with lines transporting petrochemicals. Previous regulations required owners to "demonstrate effective risk management" of their pipeline under certain guidelines that were provided by the OPS. Under existing rules, owners are responsible for conducting routine inspection of high risk pipelines. High risk is defined in terms of operating pressures and population densities. The consequence of non compliance is heavy fines by the OPS.

In the water sector, owners of water transmission mains have adopted a self managed approach to pipeline risk management. High risk areas are defined by the individual utilities and may include critical pipelines (those that feed hospitals), crossings or older areas of a system. There is a strong desire to operate lines beyond design life because of high replacement costs, however many utilities do not adequately make the connection between daily operations and pipeline risk management. In the absence of a pipeline failure, there is no perceived consequence of not effectively managing pipeline risk. Much has been made of the Government Accounting Standards Board statement 34 (GASB34) as a driver for water pipeline operator. Under new rules, utilities may choose a modified depreciation approach that would strengthen its balance sheet by allowing a maintenance expense to be tracked instead of a depreciation expense. To adopt this approach for pipelines, utilities would need to demonstrate asset management (including condition assessment) of the pipelines. Most utilities continue with a straight line depreciation of pipeline assets.

Monitoring

Third party damage is large problem in the oil and gas industry and many failures are attributed to this actively. As such, the industry invests a lot of effort in clearly marking the location of their lines, policing right of ways and in some cases to real time monitoring of pipeline corridors. While some Acoustic Emission systems may be used, monitoring is typically done using fiber optic cables buried in the right of ways to detect encroachment of the line.

In the water sector, acoustic monitoring of PCCP is performed to listen for active deterioration of the prestressing wires. This is often performed after an RFEC/TC inspection determines the baseline condition of the pipeline and then in high risk areas. While water pipelines may also be monitored by fiber optics to listen for third party damage, this is relatively rare.

Conclusion

Looking at the petrochemical and water sectors it is possible to see some general similarities in the overall approach to managing aging pipeline infrastructure. While the technologies used to assess the condition of pipelines are generally different, both sectors employ some sort of assessment technology. These technologies are generally tailored to address specific failure mechanisms of different pipe materials and are subject to the limitation in access to the pipelines (and the willingness of the industry to improve access to the pipelines since in both industries there are many miles of uninspectable pipe lines). In general there is a desire in both industries to do the minimum amount of inspection, perform the minimum amount of rehabilitation and still extend the life of the pipeline infrastructure.

While the inspection cost per linear foot is higher in the water industry, the total inspection cost is greater for the oil and gas sector. PCCP pipelines are designed to accommodate changes in operating pressure along the pipeline; however construction records tend to be much better in the oil and gas sector where steel pipeline designs are less complex. Virtually all new petrochemical lines are designed with future inspection in mind; this concept is still largely foreign in the water sector. The presence of leaks are not acceptable in any petrochemical pipeline, however they are widely accepted as a fact of life in a water system. This wide scale acceptance of leaking water pipe may change in the future. (In one Sahara® project over 180 million gallons per year were identified coming from 10 leaks in a 30-36" steel line; the retail value of the water coming from these leaks was in excess of $450,000.) Because of the huge economic and social cost of failures of petrochemical pipelines, the industry now finds itself heavily regulated. These regulations have driven condition assessment firms and operators to address the unpiggable pipelines. Because the consequence of water pipeline failures and leaks is perceived as less severe, and because few lines cross state boundaries there are presently no regulations surrounding pipe inspection or risk management in this sector. For the time being it has been left up to the industry to adopt its own best practice, however it is not clear what sort of future regulatory environment might exist.

References

Mergelas, B, Atherton, DL, Kong, X "Risk Management: A Case Study of Operating PCCP", Proceedings of ASCE Pipelines 2002, Cleveland, Ohio, USA

Mergelas, B, Bond, A, Laven, K, "Financial Benefits from Seven Years of Water Loss Control utilizing the Sahara® System at Thames Water in the United Kingdom", Proceedings of ASCE Pipelines 2006, Chicago, Illinois, USA

Mergelas, B., Jones C., Bond, A., (2004.) "Pinpointing Leaks in Water Transmission Mains", Proceedings of ASCE Pipelines 2004, San Diego, California, USA

Risk Management and Pipeline Safety Committee Presentation

"Risk Management of Pipeline Corrosion in the Water and Wastewater Industries"

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Abstract

The water utility industry has a unique perspective on corrosion and corrosion control as compared to other industries that regularly implement corrosion control, such as oil, gas or other hazardous materials. The hazardous materials industries are compelled by state, local and federal regulatory requirements to implement corrosion control in order to minimize the risk associated with failure of their piping systems and facilities. There is as yet no such compelling motivation for external corrosion control in the water utility industry. A water utility can, if it so chooses, do nothing to control corrosion, and accept the consequences. Currently the major motivating factors in the water utility industry are to provide uninterrupted water service with a high quality product, preservation of assets, reduction of operating costs (leaks), minimization of possible sources of contamination to the water supply, and pride of ownership. Corrosion leaks, just like any other pipeline "break", can result in contamination of the water supply. Reducing corrosion leaks reduces the potential for negatively affecting water quality and the possible public safety and health effect issues associated with any contamination of the water supply.

The growth of the pipe rehabilitation industry is evidence of the wastewater utility industry's dealing with the consequences of poor design assumptions regarding the risk of internal corrosion. Concrete pipes designed on the basis of corrosion rate assumptions, and force mains designed assumed to be full, for example, have had design lives significantly lessened by changes in flow regimes. Like the water industry, the wastewater industry is motivated by reduction of leaks that can lead to contamination of groundwater. The risks assumed by the utilities investing in rehabilitation of pipe and manholes lie in the assumption of design life extension of the buried assets.

Introduction

This paper is one result of the ASCE Pipeline Risk Management Committee's presentation and round-table discussion at the 2006 ASCE Pipeline Conference, held in Chicago. At that time it was apparent that the perception of many pipeline engineers that pipeline risk management is principally of concern to the high-consequence utilities, such as oil and gas, and of little concern to the water, wastewater, and reclaimed water industries. This paper is intended as an introduction to the often-overlooked risks attendant to corrosion of water and wastewater pipelines.

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Risk Management of Corrosion by Regulation as a Function of the Fluid Transported

The federal government, and many state and local governments regulate the oil and gas pipelines. The motivation for that regulation is the significant consequence of pipeline leaks. The public reaction to gas leaks, oil spills, and fuel line fires is righteous indignation. The risk of loss of any life and any environmental damage have been perceived as unacceptable, thus that perception of risk has been communicated to legislators. The public generally believes that there is enough profit being made that the cost to reduce the perceived risk of failure to nearly zero is worthy of mandate. The resulting legislation has resulted in regular corrosion monitoring, testing, and remediation requirements, solely on the basis of the fluid being transported.

An identical pipeline, made of nearly the same materials, and operating under potentially greater pressure if containing potable water will be regulated only on the basis of the *quality* of the fluid transported. Certainly water purveyors are motivated to minimize losses, but the leakage of a waterline is often accepted as a normal operating condition. Even the American Water Works Association standard for ductile iron pipe installation (ANSI/AWWA C600-99) allows leakage to levels easily achievable for rubber-gasketed joints. AWWA standards are only enforceable to the extent that the owner requires conformance thereto. The Clean Water Act as a regulation may only be influential toward reduction in risk from leakage due to corrosion when that leakage leads to reduction in water quality.

Even less exposed to regulation to minimize corrosion risks are wastewater and reclaimed water pipelines. Since the first sewers over two millennia ago, the emphasis has been on getting rid of the waste. A little leakage into the sewer has been viewed as beneficial until only recently when most wastewater has to be treated, and the additional cost to treat the inflow is burdensome. Risk of leakage out of the sewers has been largely been ignored, especially with the development of better-quality joint materials. Reliance on health department separation standards as a risk mitigation measure is nearly universal. Reclaimed water pipelines, although a recent development, are usually designed as potable water lines, but treated as sanitary sewers by health departments. And because most sewers are constructed of non-corrodible materials, such as clay, the perceived risk of failure to corrosion is low, and that risk perception is reflected in the lack of regulation.

Risk Management of Corrosion at the Design Phase

The owner's perspective to minimization of risk is founded on minimizing capital costs. Nearly all water, wastewater and to a lesser extent, wastewater utilities have regulated rate structures that provide significant impetus for the minimization of cost. The lowest capital cost perspective extends to the implicit assumption that all pipe materials are universally suitable. Although that may locally be the case, the mix of pipeline materials between projects may result in an unperceived risk of corrosion. The addition of coatings and joint bonds, and the associated maintenance costs are often perceived as unnecessary. It's not until a number of serious corrosion failures occur that that perspective changes.

The long-term use of a pipe material and its satisfactory or unsatisfactory performance may not be relevant to that product based upon current manufacturing standards. And due to geographic growth, or the merger of different systems, the performance of pipeline materials within a given utility may be entirely different from its experience with those materials due to different soil and groundwater conditions. The blanket rejection of pipeline materials may be an owner's means of risk management that is counter to its need for lowest capital cost. By reducing the number of competitors at bid, the price is less likely to be low.

The skimping on a project to minimize capital costs begins with the engineering selection. Engineers perceived by may owners as a commodity are selected on that basis, and squeezed for fees. They are motivated to design by performance specification. Design by performance specification not only includes specifying pipe materials to meet the available standards, but to specify that the engineering of the pipe is to be done by the manufacturer. Acknowledging that this is suitable for many distribution pipelines and sewer collection systems, for transmission mains and major trunk sewers the risk of corrosion (and ultimately structural) failure is perceived by the owner to be borne by the engineer's liability insurance and by the pipe manufacturer.

The Engineer's Perspective is a little different. His approach may be to rely upon the suitability of the specified standards, the reputation of the pipe manufacturer, the skill of the general contractor, and ultimately the maintenance by the owner. The perception of lowest risk is based upon the engineer's intuition, analysis, and experience. Commercial influence regarding assertions of what is standard. That perception of risk may be aided by recommendations from other professionals such as geotechnical and corrosion engineers, but ultimately the risk associated with dealing or not dealing with the probability of corrosion ultimately rests on the shoulders of the owner. Application of engineering judgment is addressed in detail elsewhere (Vick, 2002).

Performance of past materials is a significant factor in the risk assessment made, even if done without specifically considering risk. The risk of corrosion failure has affected materials selection as discussed briefly below.

Cast Iron Pipe (CIP) was initially used as a significant technological improvement over wood stave pipe, hollowed-out logs, and gravity aqueducts. The commercial development of cast iron pipe and its application to water transmission and distribution resulted in a significant improvement in public health. Improvements in joint design have reduced the leakage significantly from the old jute and lead-packed joints. Substantial evidence of long-term service has resulted in an assumption of inherent corrosion resistance of cast iron pipe that was initially extended to its successor, Ductile Iron Pipe (DIP) (ANSI/AWWA C151.) The advances in metallurgy that led to DIP also allowed a large reduction in the cost of the pipe, to the delight of capital cost-conscious utilities. Not surprisingly, DIP corrodes at about the same rate as CIP, and in corrosive soils has reportedly been disappointingly short-lived. In response to that the DIP producers introduced polyethylene encasement, as a corrosion barrier, rather than coating. Polyethylene encasement (ANSI/AWWA C105), when properly installed has been and continues to be an effective means to minimize the risk of corrosion leaks.

CIP and DIP have also been widely used for sanitary sewers and force mains. Likely as the result of widespread use of cast iron soil pipe in domestic sewer piping, unlined and uncoated CIP and DIP performance is a function not of the product but of its exposure to corrosive soils, and the amount of hydrogen sulfide gas in collection sewer systems. Force mains designed assumed to be full have had design lives significantly lessened by changes in flow regimes. (Dahlen, et al 2004) The iron pipe manufacturers now offer a variety of linings intended to protect the pipe against the acid generated within such an environment. Additional reduction in risk of corrosion can be achieved by application of polyethylene encasement.

Steel Pipe was introduced as a competitor to iron pipe, and has been widely used in diameters larger than practical to cast iron pipe. It was early recognized that the much thinner steel would be rapidly perforated, so coatings have been almost universally applied. Coal tar and cement mortar have been commercially supplemented with barrier coatings such as polyethylene and vinyl tapes, extruded polyethylene, and epoxy. Initial linings were coal tar or bitumen. Application of cathodic protection to increase the effectiveness of cement mortar coating is rarely done, but is widely applied to barrier coatings, in order to protect the steel at holidays (defects). Occasionally a utility will elect to defer installation of a cathodic protection system in an effort to reduce capital cost. Concerns about the potential toxicity of coal tar, particularly during application, have nearly eliminated its use as a lining, rather than its effectiveness in mitigating corrosion. Cement mortar lining of steel pipe (and of DIP as well) has been very successful as a means of preventing interior corrosion for water lines, but not for sewer force mains (Lippman 2004).

Concrete Pressure Pipe was also developed as a means of transporting large volumes of water in sizes not possible to achieve with iron pipe, and has been a formidable competitor to steel pipe. Particularly during the second world war and the high-growth years following, the relative scarcity and high price of steel led to widespread use of prestressed concrete cylinder pipe (PCCP) (ANSI/AWWA C301), pretension concrete cylinder pipe (ANSI/AWWA C303) and reinforced concrete steel cylinder type pipe (ANSI/AWWA C300.) These pipe offer significant potential resistance to corrosion of the steel components, being encapsulated in portland cement concrete or mortar. The pressure to remain competitive against steel and DIP resulted in design changes that may ultimately have been less conservative. Because of the relative complexity of the design, the actual pipe design was frequently specified to be the pipe manufacturer.

Some owners, in an ill-advised attempt to protect the prestressing wire added cathodic protection (CP). Sometimes that CP hydrogen-embrittled the wire, and reduced the life span. Because of the high energy contained in the prestressing wires, failure is often explosive. Coupled with the usually large diameter of PCCP, the consequence of failure has sometimes been large. Notwithstanding the development of a design procedure that is much more rigorous than any other pipe material, many utilities remain leery of its use.

Concrete pipe has also been used extensively for sanitary sewers, both un-lined, and lined. Linings that at one time were considered state-of-the–art, such as ceramic tile have proved still susceptible to sulfuric acid corrosion. At one time popular as a means of mitigating corrosion, the A-Z method of providing additional concrete lining has proved to be effective. However, once all of the concrete has reacted with the acid, the steel reinforcement is subject to corrosion. The rate of loss of concrete at least provides a means of estimating remaining service life. (Kienow & Kienow, 2004) The most successful means of protecting concrete pipe against internal corrosion is PVC-lined (e.g. T-Lok). The long-term success of that lining is dependent upon the quality of the lining … ultimately the result of capital investment in inspection.

Vitrified Clay Pipe's corrosion resistance has led it to be the material of choice for gravity sanitary sewers. Because of its inability to sustain internal pressure, other materials susceptible to corrosion have been chosen for force mains, with widely differing degrees of success. Development of Poly Vinyl Chloride Pipe (PVC), High Density Polyethylene Pipe (HDPE), and Fiberglass Pipe has provided opportunities to reduce the risks of failure due to corrosion. The use of these materials does not eliminate the risks associated with corrosion. DIP fittings and steel fabrications for these pipe materials require additional attention in design when utilizing these materials.

PVC and polyethylene pipe have been used extensively for sanitary sewer gravity (and force main) systems. These materials are generally resistant to the chemicals contained in wastewater. Illegal discharges may contain chemical compounds that will affect their serviceability. Clearly, internal and external corrosion is less significant issue with PVC and PE. However, certain chemical compounds, esters, ketones and aromatic or chlorinated hydrocarbons in certain concentrations can affect the tensile strength of the materials. Concentration of the chemical compounds and contact time are critical items to consider. The joint gaskets for these (as well as all gasketed pipe materials) will require attention during design to reduce the risks attendant when the systems are to be placed in brownfield environments, leachate collection systems, drainage, etc. Brownfield and other corrosive environments are always a risk to consider long-term effects from the in-situ chemical environment on gaskets. For any exposure, the entire system, and not just the main pipe material, determine the risk of failure. Because there is no perfect pipe material, the risks attendant to use of those non-ferrous pipe materials mentioned above should be evaluated on a case-by-case basis. It is beyond the scope of this paper to address all of those potentialities.

The best corrosion control is designed into and buried with the pipeline and can be almost "forgotten." That corrosion control need not be expensive. Coatings and linings appropriate to the soil environment and interior service must be applied to the joints in at least equal quality, or the expenditure is for naught. Leaks, just like any other pipeline "break", can result in contamination of the water supply. Reducing leaks due to corrosion reduces the potential for negatively affecting water quality and the possible public safety and health effect issues associated with any contamination of the water supply. The same benefit accrues to sanitary sewer corrosion protection.

Risk Management of Corrosion as Service Life Extension

A Call to Action: That pesky line break may be a common and accepted occurrence in a utility until its frequency or size or proximity gains the attention of the press. The unfavorable exposure may function as unintended publicity in a campaign for support of "aging infrastructure renewal", but the potential liability from property damage may exceed the funds available. Rarely are utilities motivated to protect pipelines against corrosion until a major event or events cause a reexamination of design, construction, and maintenance practices (Cromwell, et al. (2002) & Kleiner, et al. (2005)). That call to action is usually accompanied by a realization that not all of the at-risk pipe can be replaced with new pipe at either reasonable cost or without significant service disruption. Put another way, there is not enough money to make any pipeline risk-free. And once a sewer or water pipeline has been place into service it usually needs to continue in service indefinitely. That balance must come from a risk assessment process (Romer & Bell (2004); Deb, et al. (1990); & Lillie et al. (2004)) that addresses a program of monitoring and protection that extends the service life of the pipeline.

The ability to monitor a pipeline for corrosion may be the most effective means of mitigating that risk. It is not yet practical to monitor internal corrosion beyond periodic internal inspection. That ability is a function of the capital invested in the pipeline for the means of internal entry, isolation and dewatering. Tools are being developed to quantitatively evaluate the interior corrosion discovered in such examinations (Reed, et al. (2004) & Xianjie, et al. (2005)). Even internal inspection of gravity sanitary sewers can be difficult, with significant engineering judgment necessary for condition assessment based upon visual data alone. External corrosion monitoring is possible when pipe joints have been bonded, and points of connection such as test stations exist. The ability to monitor or protect without those is difficult and expensive. The literature is well populated with examples.

The choice between rehabilitation, replacement or continue to repair is not solely financial, because engineering judgment must be applied as well. The literature is rich with discussion of these choices (Marshall, et al. (2001), Mergalas, et al. (2005), Nelson, (2005), & O'Day, et al. (1989)), just to name a few. The risks assumed by the utilities investing in rehabilitation of pipe and manholes lie in the assumption of design life extension of the buried assets. Most rehabilitation methods have very short demonstrated service lives just because they are new. The qualified leap of faith in extrapolating the performance of rehabilitation methods is comparable to the design assumptions made for the original pipeline design.

There is a reported need of replacing or rehabilitating aged transmission and distribution lines and mains. There is significant publicity regarding funding of "aging infrastructure." Much of that need is a result of ignoring the potential for corrosion. Governmental Accounting Standards Board Rule 34 (GASB, 2000) established new financial reporting requirements for state and local governments by mandating tracking of asset deterioration. Pipeline operators that have not been proactive about corrosion control will experience a loss in asset value that was previously not identified. The negative impact on financial statements will limit the pipeline utility's ability to raise money to rehabilitate the pipelines. Clearly the selection of a pipe material responsive to in situ corrosive environments can reduce the probability of failure. The material can possess, inherently, the resistance necessary or can be enhanced with linings, sacrificial anodes or bagging to combat the forces of corrosion. The bottom line however, is that material selection and its enhancement is critical during design of high consequence piping systems. Corrosion cannot be ignored.

References

ANSI/AWWA C105-99, *American National Standard for Polyethylene Encasement for Ductile-Iron Pipe Systems*, American Water Works Association, Denver, CO.

ANSI/AWWA C151-02, *American National Standard for Ductile-Iron Pipe, Centrifugally Cast, for Water,* American Water Works Association, Denver, CO.

ANSI/AWWA C200-97, *AWWA Standard for Prestressed Steel Water Pipe-6 in. (150mm) and Larger*, American Water Works Association, Denver, CO.

ANSI/AWWA C300-04, *AWWA Standard for Reinforced Concrete Pressure Pipe, Steel Cylinder Type*, American Water Works Association, Denver, CO.

ANSI/AWWA C301-06 (Draft), *AWWA Standard for Prestressed Concrete Pressure Pipe, Steel-Cylinder Type*, American Water Works Association, Denver, CO.

ANSI/AWWA C302-04 *AWWA Standard for Reinforced Concrete Pressure Pipe, Noncylinder Type*, American Water Works Association, Denver, CO.

ANSI/AWWA C303-02, *AWWA Standard for Concrete Pressure Pipe, Bar-Wrapped, Steel-Cylinder Type*, American Water Works Association, Denver, CO.

ANSI/AWWA C600-99, *AWWA Standard for Installation of Ductile-Iron Water Mains and Their Appurtenances*, American Water Works Association, Denver, CO.

Cromwell, John E., III, et al. (2002), *Costs of Infrastructure Failure*, Awwa Research Foundation, Denver, CO.

Dahlen, Elizabeth P. et. al., "Failure Analysis of a Sewer Pipeline Confirms Link Between Regulatory Changes and Resultant Biochemical Impact on Pipe Materials", Pipeline Engineering and Construction, ASCE Proceedings, John J. Galleher, Jr. & Michael T. Stift, ed. (2004)

Deb, Arun K., et al. (1990), Assessment of Existing and Developing Water Main Rehabilitation Practices, Awwa Research Foundation, Denver, CO.

Kienow, Karl E. and Kenneth K. Kienow, "Risk Management ... Predicting Your Next Concrete Pipe Sewer Failure Before It Happens", Pipeline Engineering and Construction, ASCE Proceedings, John J. Galleher, Jr. & Michael T. Stift, ed. (2004)

Kleiner, Yehuda, et al. (2005), *Risk Management of Large-Diameter Water Transmission Mains*, Awwa Research Foundation, Denver, CO.

Lillie, Kevin, et al. (2004), Workshop on Condition Assessment Inspection Devices for Water Transmission Mains, Awwa Research Foundation, Denver, CO.

Lippman David R., "The Significant Results of Two Unexpected Corrosion Failures", Pipeline Engineering and Construction, ASCE Proceedings, John J. Galleher, Jr. & Michael T. Stift, ed. (2004)

Marshall, David, et al. (2001), *Managing Maintenance Costs of a Water Transmission System*, Pipelines 2001-Advances in Pipeline Engineering & Construction, Joseph Castronovo, ed., ASCE, Reston, VA.

Mergalas, Brian, et al. (2005), *Using a Combination of Condition Based Asset Management Techniques to Manage a Water Transmission System*, Pipelines 2005, ASCE, Reston, VA, pp. 782-792.

Nelson, Richard E. (2005), *Utilizing Condition Assessment Data in Pipeline Replacement and Reinvestment Planning*, Pipelines 2005, ASCE, Reston, VA, pp. 130-136.

O'Day, D. Kelly, et al. (1989), *Water Main Evaluation for Rehabilitation/Replacement*, Awwa Research Foundation, Denver, CO.

Reed, Christopher, et al. (2004), *Techniques for Monitoring Structural Behavior of Pipeline Systems*, Awwa Research Foundation, Denver, CO.

Romer, Andrew E. and Graham E. C. Bell (2004), AwwaRF, *External Corrosion and Corrosion Control of Buried Water Mains*, Denver, CO.

Vick, Steven G., Degrees of Belief – Subjective Probability and Engineering Judgment, ASCE, ISBN 0-7844-0598-0 (2002).

Xianjie, Kong, et al. (2005), Condition Assessment of Small Diameter Water Transmission Mains, Pipelines 2005, ASCE, Reston, VA, pp. 252-262.

Risk Management for Planning and Decision Making of Pipeline Projects

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Abstract

The goal of risk management is to minimize expected costs over time where the costs are defined in some probabilistic fashion. Traditionally, total expected costs due to failure are summed for planning purposes over a meaningful spatial and temporal domain and assigned a weight based on the likelihood of occurrence. Expected costs are the product of the consequence of an event (e.g. failure) and the probability of the event occurring. This is the commonly applied definition of infrastructure risk; Risk = criticality x vulnerability. The total risk-based cost is then compared with expected life-cycle costs to take mitigation action to reduce risk. Actions may include insurance, inspection, monitoring, improved construction, or increased maintenance. Action is taken if the benefits in terms of avoided costs exceed the costs associated with risk reduction.

Unfortunately the probability of occurrence of specific events is uncertain at best, and at worst the decision-maker is in total ignorance about the likelihood of an event occurring. In the face of uncertainty with respect to the probability of these events, guidance for prioritization of projects may be gained from ideas developed in portfolio investment theory, which is concerned with decision making under uncertainty. Common methods of benefit/cost-based decision criteria fail to account for uncertainty. An example risk-based calculation from the sanitary sewer field will be used to illustrate risk management at the planning level for pipeline rehabilitation projects using risk based decision criteria.

Key words: risk, asset management, life cycle, pipeline, sewer

"Without numbers, risk is wholly a matter of gut"
Peter L. Bernstein, *Against the Gods, The Remarkable Story of Risk* (1996).

1.0 Introduction

When an owner of a pipeline system makes a decision that affects the life-cycle performance of the system, some form of risk assessment is likely performed. The risk assessment may be simple and made under conditions of ignorance, where probabilities and causal factors are ignored, "assumed away", or made by "gut" experience; or the risk assessment may be based on a quantitative analysis performed under conditions of uncertainty of the underlying probabilities. Rarely, if ever, are risk assessments made under the strict conditions of formal decision theory, where all outcomes and all probabilities of each outcome are known, such as a coin toss or a die throw (Alesch et al. 2001). This paper attempts to illuminate the complex decision making issues facing pipeline owners under conditions of uncertainty with respect to risk, where "risk" is defined as the consequence of an event multiplied by the likelihood of that event occurring. Methods introduced with the goal of improving decision making come from the related but varied fields of financial portfolio theory, natural hazards mitigation investments, and asset management.

2.0 Methods

A pipeline system represents a linear, spatially distributed network asset that provides a service over a period of time. Actions taken during the life of the pipeline may improve performance or prevent loss of service, and each action comes at some cost to the owner. Actions may include inspection, performance modeling, rehabilitation, proactive preventative maintenance, and replacement. The problem facing pipeline managers is deciding which actions, if any, to take, and when to schedule them. These decisions are made under conditions of uncertainty and/or ignorance as to the types of failures that may occur, the likelihood of occurrence of the failures, and consequences of failures should they occur. Simulation tools are frequently used to examine the performance of the system under failure conditions to improve our understanding of the consequence of failure. Scenario analyses may be used to evaluate performance under failure, time series evaluations are used to estimate life-cycle costs, and Monte Carlo methods are commonly used to model probabilistic processes. The results of these analyses may be used to estimate the losses, costs or consequences of the various failures analyzed over a period of interest.

Given the complex array of tools available, it is a challenging task to first define the problem, the strategy to obtain the best information, and to implement that strategy in a decision making context. This section describes one way to address these challenges.

2.1 Risk Assessment

Risk, defined as the product of frequency and severity, also termed vulnerability and criticality, may be represented with temporal and spatial distributions. Vulnerability [V] is defined as the probability of an event occurring, and criticality [C] is the consequence of the event occurring, and the risk of the event [R] is defined as the product of [V] and [C].

2.1.1 Types of Pipeline Failure

The probability of a number of events triggering a failure may be estimated, and any number of failure definitions may be used. For example, Delleur (1994) broadly defines three different types of failures for sewer pipes: hydraulic failure, structural failure, and environmental failure. Table 1 is adapted from Delleur (1994) to distinguish between failure modes for sewer systems.

Table 1. Summary of sewer failure modes (adapted from Delleur 1994)

	Failure Mode		
	Structural	**Hydraulic**	**Environmental**
Symptom	• Subsidence • Collapse • Corrosion • Loss of soil support • Loose bricks • Soft mortar • Interference from other structures	• Flooding • Surcharge • RDII • Increased roughness • Water Hammer • Flow instabilities	• Illicit discharges to environment; including upland discharges and discharges directly to receiving waters.
Diagnosis	• Monitoring • Sewer survey (Entry/CCTV) • Non-invasive scanning (IR, radar)	• Flow monitoring • CCTV • Modeling	• Monitoring occurrence • Flow/rain monitoring • Receiving water monitoring • Modeling
Action	• Pro-active maintenance • Re-active repair • Rehabilitation or renovation • Lining • Replacement	• Maintenance • Increase capacity • Upstream flow attenuation • In-line storage • Off-line storage • Inlet control • Lining • Source control • Real Time Control (RTC) • Replacement	• In-line storage • Off-line storage • Increased treatment capacity • Inlet control • Source Control • Upstream flow attenuation • Aeration • RTC • Replacement

It is important to characterize the type of failure mode when assessing risk and the probability of failure. It is likely that a pipe may first fail by one criterion before failing in the others later in the life-cycle, resulting in multiple failures. For example a sanitary sewer may first show signs of structural deterioration, then fail hydraulically with excess Rainfall Derived Inflow and Infiltration (RDII), then fail environmentally via a Sanitary Sewer Overflow (SSO) then ultimately completely fail through structural collapse. Understanding the potential failure modes is important for managing risk because specific mitigation actions are tied to the different types of failure, and scheduling those mitigation actions during the life cycle of the pipeline may have a cascading effect on future decisions (pro-active maintenance may reduce or delay the need for future rehabilitation efforts, thus effecting the present value of future mitigation actions).

2.1.2 Diversification of risk and its effect on prioritizing risk mitigation actions

To maximize the life-cycle performance of a pipeline and the cost-effectiveness of maintenance and rehabilitation dollars spent, it is helpful to rank risk by magnitude and identify mitigation actions. To prioritize risk mitigation actions based on life-cycle costs, it is important for the decision maker to understand the expected cost resulting from a particular risk. This is turn requires an understanding of the distribution and diversification of the entire risk portfolio of the firm or municipal entity making a financial commitment. This is because the economic measure used to rank economic effectiveness of actions depend on the likelihood that the expected value of costs will be realized; a traditional benefit cost analysis may be perfectly suited to a diversified risk portfolio while being inadequate for a concentrated portfolio of risks.

It may appear at first glance that the spatial distribution of risk to a utility is an abstract and academic notion. However Allesch et al. (2002) demonstrate that this may be one critical piece of the risk mitigation puzzle that is easy to overlook. The concept itself is simple - if assets are concentrated in a way that subject the entire network to failure from a specific failure mode, the pipe assets are not diversified. If the risk portfolio is made up of diverse assets with uncorrelated failure modes, i.e. the failure modes are sufficiently independent with respect to financial losses, the risks are diversified and the firm can afford to be risk-neutral. Risk-neutrality is a condition where the decision maker neither seeks risk nor is willing to pay to avoid risk, and is interested only in the expected value of an investment and not the variance.

For diversified risk, the expected values of the long-term outcome of a specific project can be used in the financial analysis. The federal government, for example, has a diverse risk portfolio and therefore can afford to be risk neutral with respect to federal projects (Alesch et al. 2002).

> "for regulatory benefits or costs that accrue to the federal government (for example income from oil production), the federal government should be treated as risk neutral because of its high degree of risk diversification." (OMB, 1996, p.15.)

Insurance companies pay close attention to risk diversification so that expected values are realized with some confidence. With a diversified risk portfolio, the owner essentially plays "the game" many times, so variance of the likelihood of occurrence does not matter and the mean, or expected value, of losses due to failure is a meaningful measure of performance or cost.

On the other hand, many local, regional and private entities are *not* risk-diversified. A loss, break, or failure may happen once or only a few times during a life-cycle. In this case, they can not afford to be risk neutral, and must instead pay close attention to the variance of their investments because the mean value may *not* be realized.

For example, if a firm has a pipeline in a region, and the region is characterized by highly corrosive soil conditions, this firm would be wise to pay attention to the potential variance of each mitigation alternative taken to reduce the cost of corrosion because widespread corrosion could occur over the entire length of pipe. If a database of other pipelines is examined for similar soil conditions and found to have a mean failure rate of 5 breaks/mile/decade (b/m/d), with a large range realized (say maximum 20 b/m/d and a minimum of 0 b/m/d) the owner making a decision may be advised *not* to expect 5 b/m/d due to the wide variance and may be advised to understand the financial and performance implications if 15 or 20 b/m/d occur. However, if the entity owns many independent (i.e. not connected) pipelines through similar soils, then the mean might be a reasonable measure to use for long-range financial planning purposes.

2.2 Asset Management

Mauney (1999) argues that pipeline maintenance and replacement budgets tend to become more stretched as infrastructure ages, due to the accelerated failure rates of infrastructure towards the end of its functional life. The classic Weibull (also known as the “bath-tub curve”) failure distribution used in most engineering reliability analyses is characterized by a high failure rate during the burn-in phase, (also called the “infant mortality phase”). This initial period is followed by a relatively long period that is characterized by a constant failure rate, and finally there is a period of exponentially accelerated failure rates during the end of the life-cycle. For an infrastructure system, budgetary requirements that are prioritized during the long, stable period of constant failure rates will likely be inadequate during the end of the life-cycle when the failure rate begins to increase. Comprehensive financial decision making has also placed additional requirements on engineers to compete against other priorities within an organization.

Reports of engineers needing to report project priorities in financial terms are becoming more commonplace, though much of the jargon, analysis and logic behind financial decision making may be foreign to many engineers. For example, engineering measures of maintenance frequency, such as reliability, are being replaced by financial measures such as net present worth that consider costs of non-action, benefit estimation and the impact to the bottom line of an enterprise (Mauney 1999).

The lifecycle management of assets using proactive strategies is termed asset management. A vast literature has arisen in the infrastructure management field as the result of GASB 34 and various global initiatives to privatized infrastructure management. Buried infrastructure systems such as pipelines pose special problems to the proactive strategies of asset management, primarily due to the expense of inspection and difficulty in estimating deterioration rates. However, because of these issues, asset management strategies may hold particular promise to pipeline managers. Asset management provides a framework of accountability to place inspection costs in a priority list of actions that includes routine maintenance, rehabilitation and replacement.

The tools of risk and asset management are inextricably linked to pipeline decision making. Used in concert, risks to multiple types of pipeline failure may be managed in an accountable fashion, future financial outlays may be estimated with greater certainty, and overall lifecycle performance can be improved. In many ways risk management, in it's most comprehensive form, is asset management. As risks of failure are estimated over the life cycle, costs are accrued and probabilities estimated. When prioritized and used, this becomes an asset management plan.

3.0 Case study in risk-based decision making

Key elements of a case study are summarized below to demonstrate how a quantitative assessment of risk may be used in a decision making context. The case study shows how a risk-formulated problem for sanitary sewer pipeline rehabilitation may be used to find optimal levels of storage, treatment and pipe rehabilitation.

3.1 Sanitary Sewers: Vallejo Sanitation and Flood Control District

The Vallejo Sanitation and Flood Control District in Northern California collects and treats wastewater from a population of 120,000 through 438 miles of sewers. In 1999 the VSFCD responded to a consent decree to reduce Sanitary Sewer Overflows (SSOs), defined as an untreated or partially treated discharge of wastewater into receiving waters. As a result of this, VSFCD agreed to come to regulatory compliance by 2006 (Carollo Engineers and CH2MHill 2000). The principal strategies for reducing SSOs and coming to permit compliance included decreasing wet weather flow through pipeline replacement or rehabilitation, and increasing system capacity. While it was recognized that the selected strategy would rely on both flow reduction and increased capacity, it was unknown how much of each alternative would be required in the final allocation of resources to manage SSOs in a cost-effective manner.

To answer this question a risk-based analysis was done to estimate the costs and performance of each alternative over a 50 year planning period. The purpose of this study was to probabilistically estimate the failure rate (defined as SSO rate) under the no-action alternative as well as under several potential SSO mitigation alternatives. A hydrologic and hydraulic performance model was prepared for this purpose, and 50 years of rainfall input were used to estimate the SSO characteristics of the system in its current state (Carollo Engineers and CH2MHill 2000). This provided an estimate of the baseline, "do-nothing" alternative performance in terms of SSO attributes (peak, volume and timing) (Wright 2003). A database of synthetic SSO events was created using the hydrologic response model. A schematic representation of this problem is shown below in Figure 1 (Wright 2003). The problem was to determine how much to invest in pipeline rehabilitation versus additional storage and treatment rate capacity.

As seen below in Figure 1, the fundamental solutions to managing the SSOs were additional storage, additional treatment capacity, and flow reduction. Individually, each of these solutions was extremely expensive. This is due to the highly variable flow response characteristics of RDII to wet weather; while some events have extremely high flow rates, making rate-based solutions expensive (treatment and conveyance), other events have very long durations and high volumes, making volume-based solutions expensive (storage). Therefore any single alternative that must mitigate both peaks and volumes would be very expensive. RDII removal (i.e. pipe rehabilitation and replacement) may be effective in reducing both peak rates and event volume, however it is very expensive and is characterized by highly uncertain performance. While theoretically it should solve the problem, in practice it often does not. This is usually due to ineffective rehabilitation plans that do not, or cannot, rehabilitate privately owned sewer laterals, a common source of RDII.

It was clear from an examination of the magnitude of flows and unit costs of individual solution elements, that a cost effective management plan would contain each of these fundamental solution elements. However the question remained how to select the appropriate balance of solutions. The answer must be justifiable to meet financial limitations as well as have a high degree of confidence that the solution would reduce SSOs by 2006 to meet regulatory and judicial requirements (Wright 2003).

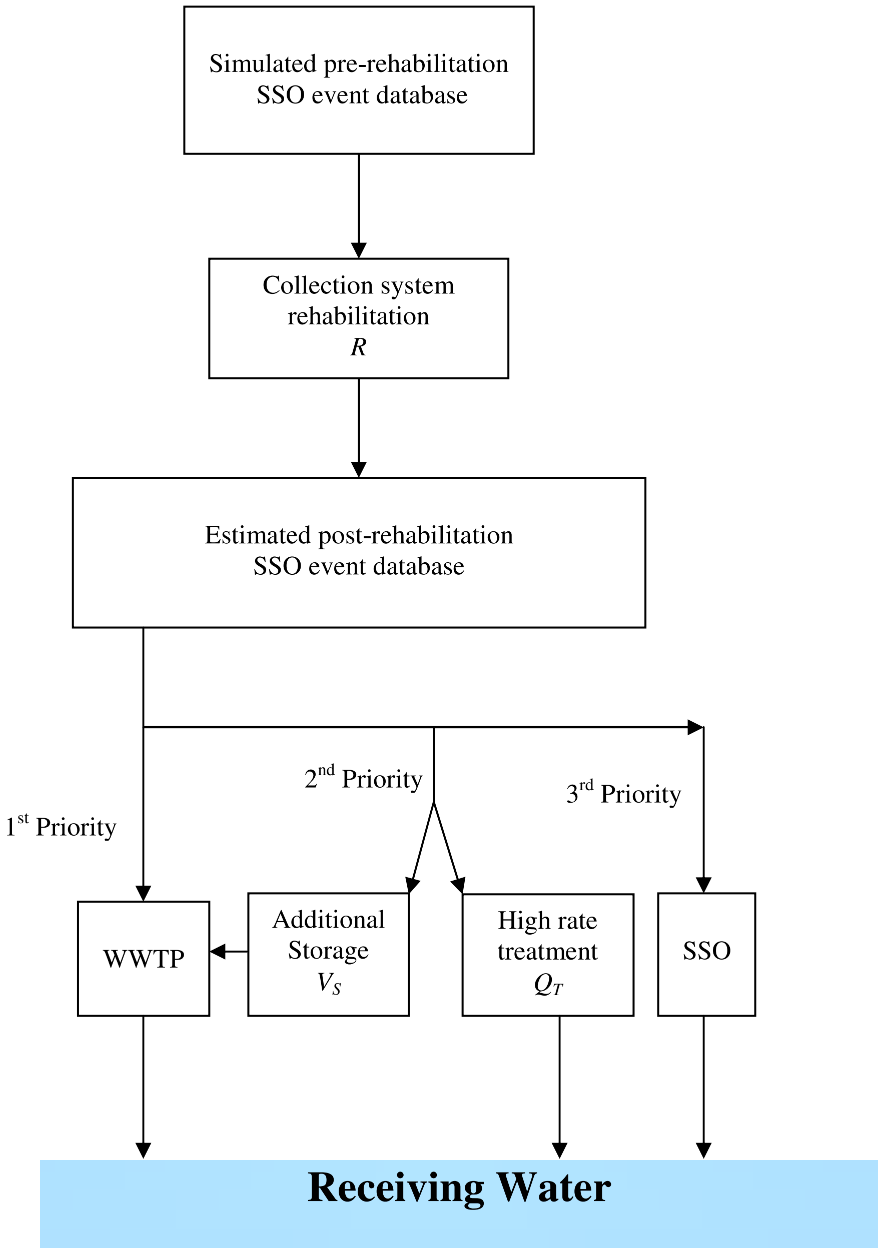

Figure 1. Schematic representation of the general SSO mitigation problem (Wright 2003).

A database of 50 years of synthetic SSO discharges was used in an economic optimization model to simulate the performance in terms of reduced failure rate of a large number of possible combinations of storage, treatment, and rehabilitation. Performance uncertainty for storage, treatment and rehabilitation was modeled using Monte Carlo sampling of underlying distributions. Uncertainty in the influent BOD concentration was modeled using a distribution based on measured data. This was important because rehabilitation removes ”clean” RDII water, therefore wastewater concentration is affected by rehabilitation; concentration increases if rehabilitation is effective. Treatment efficiency is also reduced with dilute wastewater, therefore the effect of rehabilitation on wastewater strength and treatment efficiency is tied to rehabilitation effectiveness.

The primary source of uncertainty in the model however was in estimating the effectiveness of rehabilitation at removing RDII from the system. Due to the widely dispersed nature of RDII sources in the systems, the dependence of the response to rainfall on a complex groundwater reaction, and the effectiveness of the rehabilitation technique itself, it is impossible to accurately predict *a priori* what the post-rehab RDII response would be for any given investment in rehabilitation. RDII removal from similar systems was examined and a wide variance in performance was noted. For example, effectiveness was shown to range from approximately 28% to 76% in peak flow reduction. These data are shown below in Figure 2. This widely scattered data was the basis for using a simple triangular distribution in the Monte Carlo risk assessment model.

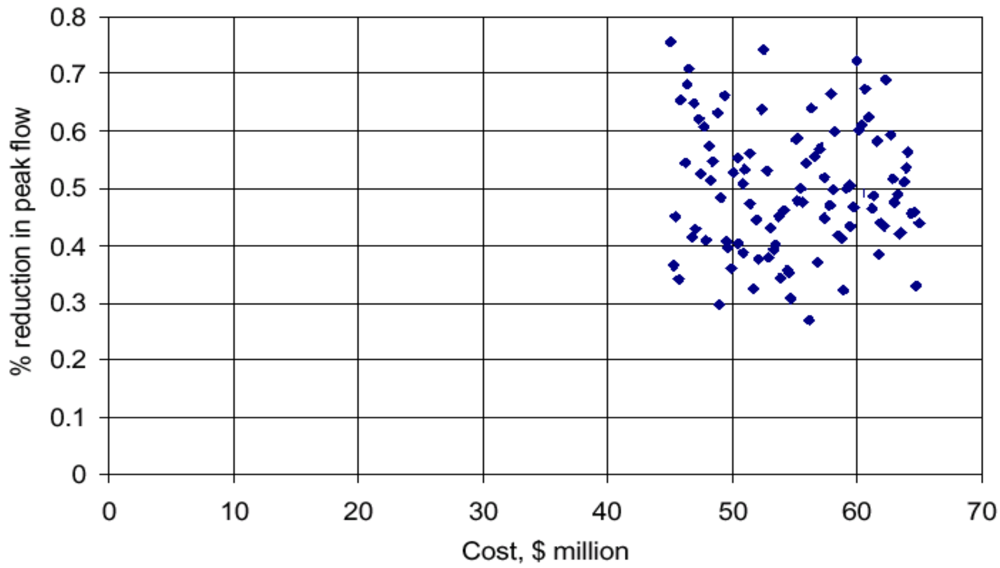

Figure 2. RDII removal rates from similar rehabilitation projects (Carollo and CH2MHill 2000).

3.1.1 Performance, optimization model, and risk analysis

Wright (2003) describes the optimization and risk analysis model used to develop solutions for this system. A genetic algorithm was used to optimize the system using cost functions of the storage, treatment, and rehabilitation options. Within the genetic algorithm, a Monte Carlo routine was used to sample the distributions described above. This model was constructed in Excel using the "Risk optimizer" software package by Palisade Corp (2000).

Cost functions were based on real data obtained from local projects. The cost of rehabilitation as a control option was based on rehabilitation projects conducted within Vallejo's collection system. These data are given below in Table 2. The cost functions for storage and expanded treatment capacity are given in equations 1 and 2 respectively.

(1) $C_S = 17{,}032\ V^{0.85}$
where C_S = the life-cycle total costs of storage in Millions US \$
V = Storage Volume (CM)

(2) $C_{HRT} = 84.5\ Q^{0.785}$
where C_{HRT} = the life-cycle total costs of high rate treatment in Millions US \$
Q = Maximum high rate treatment rate (l/s)

3.1.2 Solutions

The model demonstrated that if rehabilitation were effective, it could be a cost effective solution for managing SSOs. However the uncertainty was far too great to rely on this solution to meet regulatory and judicial mandates, while have a high probability of success in such a short time schedule. The probability of failure coupled with the consequence of failure (risk = probability x cost) was far greater than the "downstream" capacity increases in storage, conveyance and treatment. And if the rehabilitation program proved ineffective at meeting permit requirements, the expense of fines and additional rehabilitation would have been exorbitant. The optimizer selected a reasonable mix of storage and treatment to meet the regulatory requirements.

The large database of solutions generated with the model was used to create iso-quant lines of equal long term BOD removal. The effect of risk, in terms of the probability of meeting a particular BOD removal rate (as this is a function of SSO discharge, this is a measure of risk for any particular solution) was then used to compare solutions. Percent BOD removal from SSOs for the 48% and 55% isoquants are shown in Figure 3, along with the amounts of pipe rehabilitation, storage and high rate treatment required to meet the treatment level (Wright 2003).

Table 2. Cost data obtained from test basin rehabilitation project in Vallejo CA (Wright 2003).

DESCRIPTION	PILOT AREAS II	PILOT AREAS III	PILOT AREAS V
AREA TYPE	Residential	Residential	Urban
PIPELINE MAINS			
Total Length (m) (1)	7,058	3,796	4,498
Rehabilitated Length (m) (1)	5,016	3,701	3,579
Percent of Pipe Rehabed	*71%*	*98%*	*80%*
Total Costs (2)	$1,455,000	$900,000	$1,294,000
Unit Cost: Pipeline main ($/m)	$290	$243	$362
LATERALS			
Total Lateral Connections (1)	410	260	217
UPPER			
Rehabed Upper Laterals (1)	186	185	40
Percent of Upper Laterals	*45%*	*71%*	*18%*
Total Costs (2)	$383,000	$686,000	$97,060
Unit Cost: Upper Lateral	$2,100	$3,700	$2,400
LOWER - Backyard/Alley			
Rehabed Lower Laterals (1)	82	226	43
Percent of Lower Laterals	*20%*	*87%*	*20%*
Total Costs (2)	$137,000	$438,000	$79,000
Unit Cost: Lower Lateral - Backyard/Alley	$1,700	$1,900	$1,800
LOWER - Street			
Rehabed Lower Laterals (1)	161	30	128
Percent of Lower Laterals	*39%*	*12%*	*59%*
Total Costs (2)	$274,300	$58,000	$246,000
Unit Cost: Lower Lateral - Street	$1,700	$1,900	$1,900
Total Costs (3)	**$2,249,000**	**$2,080,000**	**$1,716,000**

Comparison of the variance between different solutions found with the GA optimizer is also possible using a Monte Carlo risk assessment. This comparison is shown in Figure 4. This demonstrates the deviation in performance (measured in terms of BOD removal – a function of SSO mitigation) from the expected value. Two example solutions, one for 66% expected removal (43,500 CM of storage, 78.8 l/s treatment and zero rehabilitation at a total cost of $57 M) and one for 70% BOD removal (45,400 CM of storage, zero high rate treatment and 40,000 feet of rehabilitation for a total cost of $58 M) show that for an additional $1 M, a better than expected benefit is possible. This is shown by the larger difference between the two lines at the top end of the curve compared to the bottom. This measure of risk of failure shows that there is more "upside" to the 70% solution than downside (performance below the mean). An examination of the difference in the two solutions indicates that rehabilitation – if it works, will do a better job in the long term than the high rate treatment option for this application.

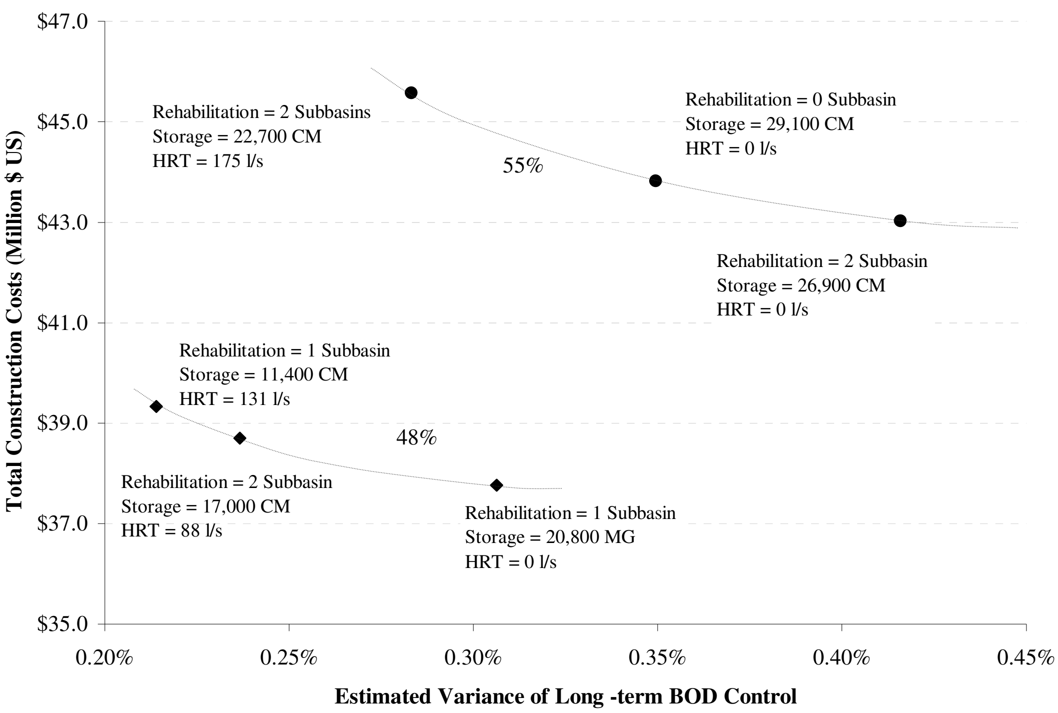

Figure 3. Example output of risk-based design decision model (Wright 2003)

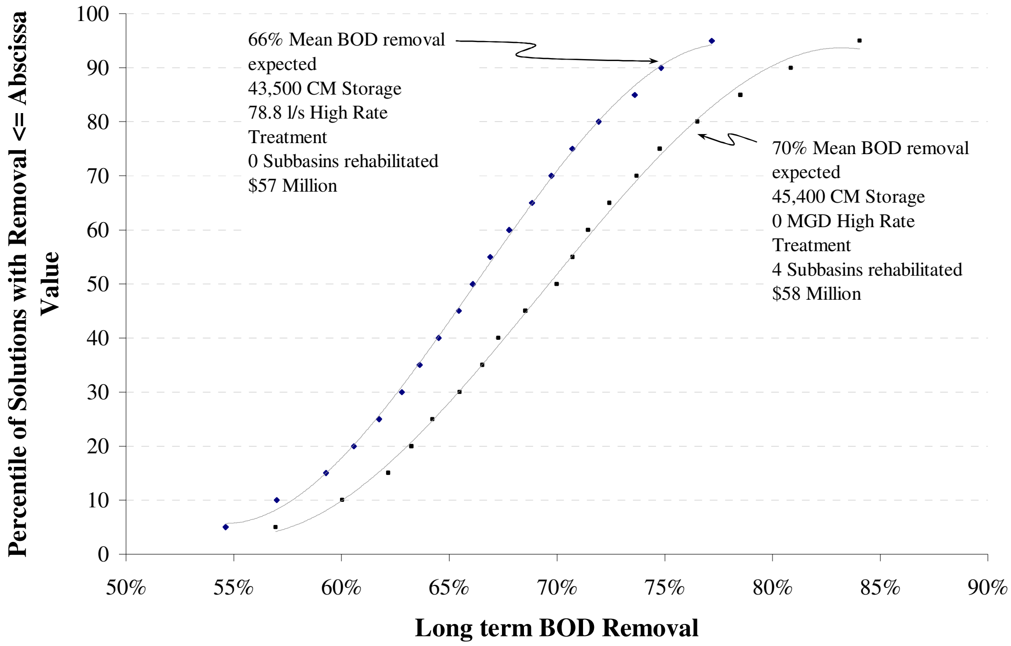

Figure 4. Results of Monte Carlo risk assessment of two example solutions (Wright 2003).

4.0 Conclusions

From a risk-based perspective, rehabilitation as the sole solution, or as a primary solution, was non-diversified in both space and time – if the solution failed to satisfy system-wide permit standards in the time allotted, the costs could be ruinous. For example, from the data shown in Figure 2 above, the expected value of the performance (say roughly 55%) could not be used as an indicator of expected performance. Instead the decision makers correctly identified the potential disastrous results that could have occurred had a significant investment been made in rehabilitation and only a 30 % reduction in the peak been realized.

The "downstream-focused" solution is strongly a function of the short time-line mandated by the courts. Absent this time line, perhaps a more proactive, holistic and rehabilitation-based solution would have proven cost effective. Referring above to the modes of sewer failure in Table 1, it is likely that a significant portion of the pipes that were currently failing under the environmental and hydraulic criteria would soon fail structurally and need to be replaced anyway. With this in mind, a true cost-based solution should have included these costs – i.e. the downstream storage and treatment-based solutions should still include some pipe replacement costs as the pipes began to structurally fail. The rehabilitation (or replacement in some cases) based solutions would delay or explicitly include these costs – thereby making them much more competitive.

5.0 References

Alesch, D.J., Nagy, R. and Taylor, C. (2001). "Seeking criteria for the natural hazard mitigation investment decision". *Acceptable Risk Processes: Lifelines and Natural Hazards*, Taylor, C. and VanMarcke, E. (eds). ASCE, 1801 Alexander Bell Dr., Reston VA 20191-4400.

Carollo Engineers and CH2MHill (2000). *Engineering Feasibility Study, Volumes 1 through 3*. Vallejo Sanitation and Flood Control District, Vallejo , CA. CD-ROM.

Delleur, J.W. (1994). "Sewerage failure, diagnosis and rehabilitation". Urban Drainage Rehabilitation Programs and Teechniques, Macatis, W.A. (ed). ASCE, 1801 Alexander Bell Dr., Reston VA 20191-4400.

Levy, Haim, and Sarnat, M. (1984). Portfolio and Investment Selection: Theory and Practice. Prentice Hall International, NJ.

Lippai, I., and Wright, L.T. (2005). "Criticality analysis case study: Zone 7 water distribution system". *ASCE Pipelines 2005: Optimizing Pipeline Design, Operations & Maintenance*. August 21-24, Houston TX.

Mauney, D. 1999. "Financially optimizing the timing of major equipment maintenance against the consequence of lost reliability". *8th International Process Plant Reliability Conference*, Houston, TX. Society for Maintenance & Reliability Professionals, 8201 Greensboro Drive, 3rd Floor, McLean, VA 22102.

Palisade Corp. (2001). *Risk optimizer* (CD-ROM), Palisade Corp, Newfield, N.Y.

Wright, L.T. (2003). *Design Optimization of a Sanitary Sewer Overflow Control Plan.* Ph.D., University of Colorado, Boulder, CO. Dissertation Advisor: Professor James P. Heaney.

Technologies to Assess and Manage of Providence Water's 102" PCCP Aqueduct

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Abstract

In 1996, Providence Water experienced a catastrophic failure of its 102" PCCP aqueduct pipeline. Subsequently, the main underwent an extensive assessment and repair and was returned to service with plans that the main would be re-inspected in approximately 5 years.

In 2005, Providence Water re-inspected the aqueduct. Since the previous inspection, the state-of-the-art for assessing PCCP mains has progressed significantly. Non-destructive technologies available for assessing and monitoring PCCP pipe have made significant strides. Providence Water implemented state-of-the-art inspection procedures to obtain the best possible assessment of the aqueduct. Following the assessment of 4.5 miles of the aqueduct, Providence Water opted to install a fiber optic acoustic monitoring sensor to continuously monitor the condition of the aqueduct and identify pipe sections experiencing ongoing wire break activity.

Providence Water utilized the following technologies during the most recent 2005/2006 inspection/monitoring program:

- Electromagnetic Inspection- to detect wire breaks in the prestressing wire
- Visual and Sounding Inspection- to inspect for cracks or delaminations
- Resistivity Testing- to determine the actual number of wire breaks on excavated pipe sections (vs. the estimated number based on the electromagnetic inspection)
- Acoustic Monitoring- to detect future wire breaks as they occur in the operational aqueduct

Following the initial inspection, one pipe section was found to be in a state of incipient failure. As a result, several nearby pipe sections were strengthened and a decision was made to install the acoustic monitoring system. This paper focuses on the assessment and monitoring technologies used during this project and describes the capabilities and limitations of these technologies.

Providence Water Supplemental Tunnel and Aqueduct (STA)

Providence Water operates a 9.5 mile tunnel/aqueduct system to convey potable water from a treatment plant to the distribution system. This system is one of two aqueducts that together deliver an average of 70 mgd of water from Providence Water's sole treatment plant source to the many towns and municipalities which it serves.

The aqueduct portion of the STA was constructed in the 1960's and consists of a 5.0 mile long 102-inch prestressed concrete cylinder pipeline (PCCP) and a 3.8 mile long 78-inch PCCP. The remainder of the STA consists of reinforced concrete tunnels and shafts.

The portion of the STA addressed by this paper is the 102-inch PCCP section. This portion of the aqueduct was built in the late 1960s. Its design included 15 different pipe classes with a core thickness of 6.5 inches. Prestressing wire pitch varied from 15 to 30 wraps per foot. The pipe consisted of both mortar coated and concrete coated pipe sections. The pipe was supplied by the Interpace Corporation, but predates the Class IV wire issue, as the wire used was Class II.

Failure of the 102-Inch Aqueduct and 1998 Inspection

On November 17, 1996, a section of the 102-inch aqueduct ruptured due to corrosion, leaving several communities with limited potable water supply until repairs were completed. The pipe that failed had a concrete coating and it is believed that the coating separated from the concrete core exposing the prestressing wires to a corrosive environment.

In 1998, an internal inspection was performed of the 102-inch aqueduct to assess its structural integrity. This inspection utilized state-of-the-art inspection techniques at that time and relied on visual inspections, hammer soundings, and impact echo testing. The inspection identified several pipe sections which had loose portions of concrete coating. This inspection led to the carbon-fiber repair of 27 pipe sections, one of the first applications of carbon fiber repair in PCCP.

Following the inspection and repair work, it was decided that the main would be re-inspected in approximately 5 years.

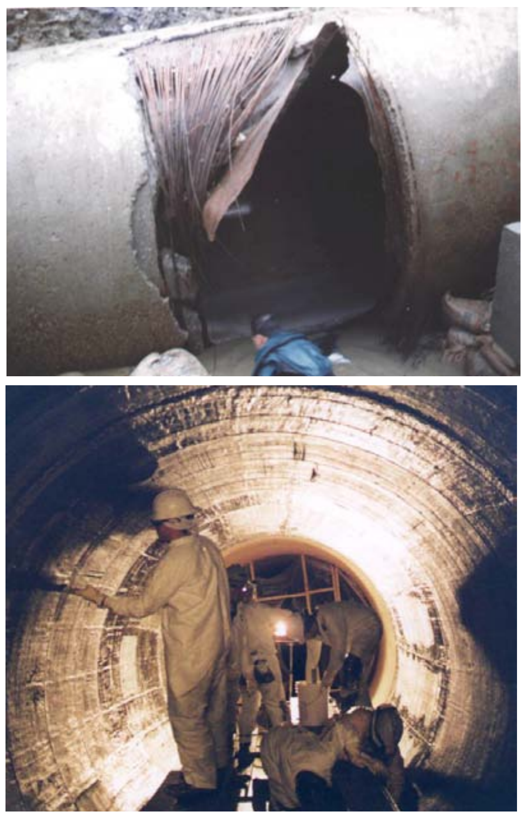

Figure 1: 1996 failure of 102-inch PCCP aqueduct and 1998 carbon fiber repairs

Providence Water Inspection 2005/2006 Program

Providence Water initiated the re-inspection of the aqueduct in 2005. Similar to the 1998 inspection, the assessment was to use state-of-the-art assessment tools for PCCP. However, techniques to assess PCCP had progressed significantly since the previous inspection and it was decided to use other techniques during the 2005/2006 inspection program. During the execution of the inspection, the following assessment techniques were utilized:

1. Electromagnetic Inspection
2. Visual and Sounding Inspection
3. Resistivity Testing
4. Acoustic Monitoring

All of the techniques are generally focused on ascertaining assessment data on the condition of the prestressing wire, the component of the pipe that provides its strength and that can be vulnerable to corrosion.

Figure 2: Electromagnetic inspection of the 102-inch Providence aqueduct

Electromagnetic inspection is a non-destructive testing technique that involves traversing the interior of a pipeline with equipment that can electromagnetically test the condition of the prestressing wire wrapping in a PCCP. An electromagnetic field is induced on the prestressing wire on one side of the pipe. Using the prestressing wire, this field is transmitted to the opposite side of the pipe where it is measured and recorded. The data is then evaluated to identify electromagnetic anomalies consistent with wire breaks in the prestressing. These anomalies are further evaluated, to determine the spatial characteristics of the anomaly and estimate the length of damaged pipe and number of wire breaks.

Visual and sounding inspections are non-destructive techniques which involve visually inspecting and sounding the interior pipe wall for evidence of a loss of prestressing in a pipe. Certain types of cracks are indicative of a loss of prestress, which is an indication that the pipe section has significant wire break damage. The sounding portion of the inspection involves tapping the interior surface of the pipe with a hammer or bar to listen for “hollow areas” indicative of a delamination. Delaminations are often associated with significant wire break damage.

Resistivity testing is a technique that can be used on an individual pipe section to ascertain an accurate number of wire breaks for that pipe section. It requires an excavation to the crown of the pipe and a chipping hammer to expose the prestressing wire. A resistance meter is then used to measure the resistance of a prestressing wire loop(s). Intact wires have a low resistance, but where a loop has an abnormally high resistance value, the loop is broken. This is an accurate method of determining the number of wire breaks on a particular pipe section. Subsequently, the pipe is patched with mortar.

Acoustic monitoring is a non-destructive monitoring technology that relies on continuously monitoring the acoustic activity propagating through a pipeline to identify the acoustic event associated with a breaking wire (identifying the snapping sound). This technology provides the time and location of wire breaks for a pipeline and is useful for determining which pipe sections are experiencing active wire break activity and what the rate of activity is (i.e. number of wire breaks on a pipe section per month).

Figure 3: Resistivity testing being performed on a 54-inch pipe

Execution of the Inspection

For logistical reasons, the inspection of the 102-inch was broken down into two sections. A 4.5 mile portion of the aqueduct was inspected in October 2005 and a 0.5 mile portion was inspected in December 2006. The inspection personnel consisted of two teams: an electromagnetic inspection team (Pure Technologies) and a visual and sounding inspection team (CDM). Both teams traversed the pipe together.

For the 4.5 mile inspection, the electromagnetic inspection team pointed out suspected electromagnetic anomalies indicative of wire break damage as the data was gathered. Although reliably identifying anomalies requires a more detailed analysis, pointing out suspected electromagnetic anomalies during this first view gave the visual and sounding inspection team, the opportunity to closely inspect numerous pipe sections that were later confirmed to have electromagnetic anomalies consistent with wire break damage. For pipe sections suspected of exhibiting electromagnetic anomalies, the visual and sounding inspection team immediately performed a close-up visual inspection and hammer sounding.

Pipe Section 5BH9

Once the field work for the inspection was complete, the intent of Providence Water was to return the aqueduct to service. However, the inspection identified one pipe section, Pipe Section 5BH9, as having a serious problem. This pipe section was located in the vicinity of the 1996 failure.

During the electromagnetic inspection, 5BH9 had a significant electromagnetic anomaly consistent with wire break damage. An electromagnetic calibration for Providence Water’s transmission main was not feasible for the 102-inch STA. Calibrations provide data on the nature of electromagnetic anomalies caused by wire break damage and improves the accuracy of the data analysis and results. Without a calibration, Pure Technologies relied on past calibrations and inspection experience for similar pipes to evaluate the Providence electromagnetic data. This analysis indicated that there was an area of wire break damage, consisting of approximately 45 wire breaks, located approximately 4.2 feet from the upstream joint.

During the visual and sounding inspection, a four foot long hollow area was identified on this pipe section. There were no cracks visible on the pipe section. The hollow area did not match with the location of the electromagnetic anomaly as it was located in the quadrant of the pipe closest to the downstream joint.

Given that this pipe section exhibited an electromagnetic anomaly and a significant hollow area, it was recommended that the aqueduct not be returned to service until the pipe was further evaluated and repaired. The pipe was then excavated and the exterior of the pipe was sounded. Nearly the entire length of the coating on one side was hollow. The coating was removed and numerous wire breaks were observed. In fact, the pipe section had more broken wire wraps than intact wraps and the pipe section was obviously in a state of incipient failure. This pipe section was repaired with post-tensioning tendons. Furthermore, given the close proximity of this pipe to the 1996 failure and the relatively high consequence of failure in this area of residential homes and apartments, it was also decided to carbon fiber repair seven pipe sections in this area.

Figure 4: Excavation of Pipe section 5BH9. Upper left- overall photograph of excavation; lower left- crack in mortar; upper right- photo with mortar coating partially removed.

One of the interesting findings on Pipe Section 5BH9 is that the electromagnetic anomaly for this pipe section appeared at the location of intact wires instead of broken wires. This electromagnetic phenomenon can occur when there is extensive wire break damage on a pipe section. Therefore, as was the case with Providence Water, it is usually advisable to combine electromagnetic inspections with visual and sounding inspections. The two inspection techniques are complementary and address limitations in each technique.

Figure 5: 2006 carbon fiber repairs

Other Pipe Sections with Electromagnetic Anomalies

Besides 5BH9, there were 50 other pipe sections that were identified to have electromagnetic anomalies consistent with wire break damage. Wire break estimates on individual pipe sections ranged from 5 to 70 wire breaks.

To provide calibration data for the electromagnetic inspection analysis and to determine the actual condition of several at-risk pipe sections, Providence Water excavated twelve pipe sections initially reported to have electromagnetic anomalies to obtain the actual number of wire breaks through continuity testing. Based on this calibration data, it was determined that one class of electromagnetic anomaly was caused by a phenomenon other than wire break damage. Also, it was learned that the algorithms used to estimate wire breaks were overestimating wire break damage. Using this calibration information, the algorithms were modified, which resulted in a net reduction in the estimated number of wire breaks.

The original wire break estimation algorithms were based on PCCP with shorting straps. Shorting straps are used (primarily on the west coast) during manufacturing to make all wire electrically continuous for the provision of cathodic protection. However, the STA does not have shorting straps. At around the same time as the Providence inspection, several other calibrations on east coast pipes without shorting straps were performed, including Greater Lawrence Sanitary District (72-inch), Springfield, MA (60-inch), Washington Suburban Sanitation Commission (60 and 96-inch), and the City of Baltimore (54-inch). Calibration data from the STA is similar to these other calibrations and it has been learned that the effect of a relatively few number of wire breaks (e.g. less than 10) creates a relatively large (as compared to the amount of damage) electromagnetic anomaly. As damage grows from this point, the size of anomaly also grows and tends to approach a similar relationship for both types of pipe.

With the resistivity testing and calibration complete, it was concluded that a pipe section with 44 wire breaks had the highest wire break total. This total was arrived at through the resistivity testing and it was learned that the wire breaks were scattered along the pipe section. This pipe section and all other pipe sections were deemed to be at an acceptable level of risk, so it was decided that the 102-inch STA could be returned to service. However, given the past history on the main and that 50 other pipe sections have electromagnetic anomalies consistent with wire break damage, Providence Water opted to install a long-term continuous acoustic monitoring system to track the performance of the aqueduct.

Long Term Acoustic Monitoring

Acoustic monitoring systems for PCCP mains are available in a variety of configurations. The types of sensor and communication protocols vary, but all designs must continuously monitor acoustic activity in a pipeline to detect the acoustic event associated with a wire break.

Figure 6: Steel strap and strain relief connection used to attach fiber optic sensor to the interior pipe wall of the 102-inch aqueduct.

For long term monitoring of long mains, fiber optic sensors are usually most cost effective. These sensors consist of four or more glass fibers bundled with a strength member and encased in a protective sheathing. The sensor is run along the invert of the pipe and attached to the pipe as required. A laser is used to project light down the fiber and a data acquisition system monitors reflections generated by the acoustic activity in a pipeline. The advantages of this technology are:

1. No electronics are placed in the water flow;
2. The entire fiber acts as a sensor, so the sensor is never further than a pipe diameter from a wire break,
3. Long sections of pipe (up to approximately 12 miles) can be monitored with one data acquisition system;
4. Monitoring system noise (e.g. electronic noise) is nearly eliminated.

For these reasons a fiber optic sensor was used to monitor the 102-inch aqueduct. The sensor was installed on the invert of the aqueduct and the main was re-charged. The wire breaks recorded by the acoustic monitoring system are now added to the estimated wire breaks determined by the electromagnetic inspection and thus at any point in the future, Providence Water can ascertain an estimated number of wire breaks on each pipe section. This total number of wire breaks can be entered into a structural analysis model and the risk associated with each pipe section can be calculated. If a pipe section deteriorates to an unacceptable level of risk, Providence Water can intervene to initiate a repair of the pipe section thus avoiding pipe failure.

Conclusions

Providence Water performed a state-of-the-art assessment of the 102-inch portion of the STA aqueduct and tunnel system. The assessment identified one pipe section in a state of incipient failure and 50 other pipe sections that had electromagnetic characteristics consistent with prestressing wire break damage. The pipe section near failure was repaired and the remaining pipe sections were deemed to be within an acceptable level of risk.

A long term acoustic monitoring system was installed in the aqueduct to monitor wire breaks into the future to provide a proactive management solution for Providence Water. The wire breaks recorded by this system are added to the wire break estimates calculated during the electromagnetic inspection results, which provides Providence Water with the ability to estimate how many wire breaks are on each section of pipe in near real time.

The implementation of these inspection techniques for Providence Water required an understanding of the fundamentals and limitations of each technology. The assessment program was successful because it was structured to use multiple assessment techniques to address the limitations of any single technique.

Assessment of residual tensile strength on cast iron pipes

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Abstract

Criteria for rehabilitation priority are an important factor in planning the improvement of water supply systems. In this study, deteriorations of water pipes are discussed to evaluate structural stability of deteriorated cast iron transport and transmission pipes. For the purposes, safety factor is introduced and estimated by measuring tensile strength and analyzing stress caused by the internal-external loads working on buried pipe body. Prediction models of a residual tensile strength are presented using pit characteristics and fracture toughness.

Related information is surveyed and collected in the fields up the pipe by digging and assessing its structural stability.

. The collected data is analyzed to evaluate the deterioration degree of the pipes including corrosion and pitting characteristics, fracture toughness, installation environment and water qualities etc. Assessment models developed in this study showed a little correlation for measured residual tensile strength. The results will very help water utilities to manage water pipes in the aspect of rehabilitation and assessment of structural safety.

Introduction

The state of water pipes is affected by multiple factors such as, exposed time, installation environment, operation conditions etc. Corrosion is the most influential factor. Corrosion decreases the thickness of water pipe and reduces the resistance to the internal and external loads.

Several researchers have studied and assessed the effects of corrosion of water pipes(Rajani et al., 2000, Rajani B. and Maker, 2000; Atkinson et al, 2002; Seica et al., 2000, 2004).

The decrease of pipe strength was evaluated due to the corrosion and pitting of deteriorated water pipes by mechanical tests and empirical models are proposed for the prediction of residual tensile strength. Two approaches have been used for the prediction of residual tensile strength. The first method is using statistical relations between mechanical intensity and geometric characteristics of corrosion pitting. The second approach makes use of the fracture toughness. Through the prediction of residual strength, it is possible to assess the safety factor. Safety factor is applied to the assessment of structural stability of water pipes, which is expressed by the ratio of residual strength and stress by internal and external loads. Application of safety factor has the advantage in prediction of failure of risk by the estimation of the stresses based on the internal and external loads.

In this study, the strength losses of cast iron pipes are analyzed by estimating the geometric characteristics of corrosion pitting and experimental models are proposed. And the prediction models of residual tensile strength are presented by an evaluation of fracture toughness.

Methodologies

The samples of cast iron pipes(CIPs), CIPs are collected in N, G water distribution systems installed between 1970 and 1983. The diameters range from 80 mm~100 mm. The characteristics of samples can be seen in Table 1.

Table 1 The characteristics of samples

No.	Specimen name	Age of pipe (years)	Nominal diameter (mm)
1	NS-C-1	1970	80
2	GM-C-3	1974	200
3	GM-C-4	1974	150
4	NS(2)-C-1	-	150
5	NS(2)-C-2	-	250
6	GM-CR-1	1974	150
7	NS-CR-1	1975	200
8	NS-CR-3	1983	150
9	NS(2)-CR-1	-	200

Test of tensile strength

Tensile strength of a material is tested by ASTM E8-96a (ASTM 1996a). In this study, tensile strength test is performed and the residual strength of CIPs samples is evaluated using specimen coupons shown in Fig. 1.

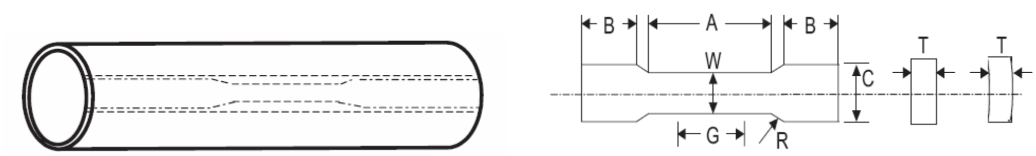


Figure 1 Specimen of longitudinal tension test (ASTM E8-96a) and the shape of flat coupon for the test.

The tensile strength can be calculated Eq.(1) by dividing breaking loads,(W_b) into gross area including pit corrosion (A_g).

$$\sigma_{rts} = W_b / A_g \qquad \text{Eq.(1)}$$

Where, σ_{rts} = residual tensile strength, kgf/mm^2
W_b = breaking loads, kgf
A_g = Gross area including pit corrosion, mm^2

To evaluate the effects of the strength loss by pit corrosion on tensile strength, net metallic tensile strength are estimated by Eq. (2).

$$\sigma_{nts} = W_b / A_n \qquad \text{Eq.(2)}$$

Where, σ_{nts} = net metallic tensile strength, kg$_f$/mm^2
W_b = breaking loads, kgf
A_n = net metallic area, mm^2

Loss of strength can be calculated by Eq.(3)

$$\text{Loss of strength} = (\sigma_{rts} - \sigma_{nts}) / \sigma_{nts} \qquad \text{Eq.(3)}$$

Test of fracture toughness

Fracture toughness of a material is tested by ASTM E399(ASTM 1990). It is reported to be hard to manufacture specimen of CIPs, DENT(Double-edge notch tensile) was used as specimen by simple tension test(Rajani et al., 2000). The specimen can be seen in Fig. 2.

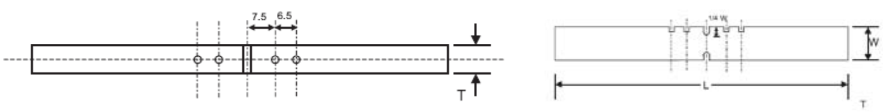


Figure 2 DENT specimens for fracture toughness test

The fracture toughness is calculated by following expression Eq.(4) by using measured data.

$$K_q = \beta \sigma_n (\pi \alpha_n)^{0.5} \qquad \text{Eq.(4)}$$

Where, σ_n = Nominal stress
α_n = length of notch
b = geometric factor by the shape of specimen

Nominal diameter, specimen width, gross thickness, average pit depth, net metallic thickness, gross area, net metallic area of each specimen can be shown in Table 2.

Table 2 Pit characteristics of tensile specimens

ID	Nominal diameter, mm	Specimen width, mm	Gross thickness, mm	Avg. pit depth, mm	Net metallic thickness, mm	Gross area, mm^2	Net metallic area, mm^2
NS-C-1	80	12.62	9.57	6.49	3.08	120.77	38.76
GM-C-3	200	12.73	11.92	3.49	8.43	151.74	106.64
GM-C-4	150	12.64	10.10	4.10	6.01	127.66	75.54
NS(2)-C-1	150	12.46	11.95	1.42	10.54	148.90	131.16
NS(2)-C-2	250	12.48	16.35	3.23	13.12	204.05	163.74
GM-CR-1	150	12.50	9.07	0.63	8.44	113.38	105.37
NS-CR-1	200	12.61	9.60	1.73	7.87	121.06	99.02
NS-CR-3	150	12.60	9.62	1.58	8.04	121.21	101.34
NS(2)-CR-1	200	12.55	15.00	1.67	13.33	188.25	166.34

Loads-strain relationship are analyzed as shown in Fig. 3 and relationship between breaking loads and gross or net metallic thickness can be seen in Fig. 4.

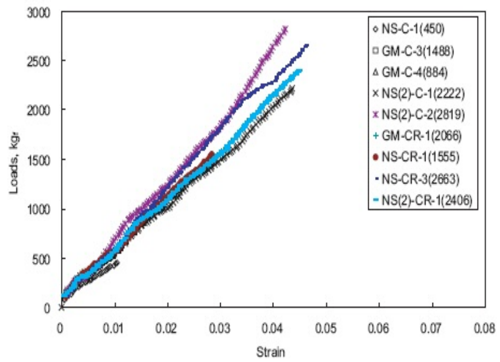


Figure 3 Loads-strain plot of each specimen.

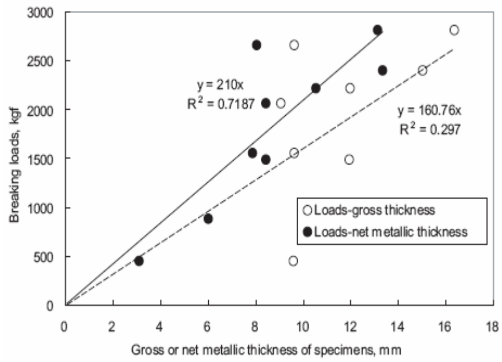
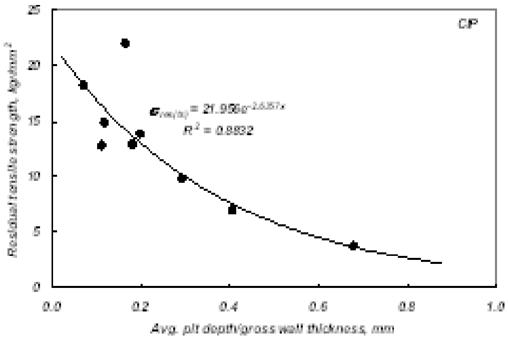


Figure 4 Breaking loads versus gross or net metallic thickness.

Prediction of residual strength

Residual strength is expressed as the characteristics of pitting corrosion, the ratio of pitting depth and initial thickness as shown in Fig. 5 and it is found that tensile strength decreased exponentially as pitting depth is increased. At known pitting depth, it is possible to estimate residual strength by using this relation.

The relation curve can be seen in Fig 5, which describes the relationship between pit characteristics (p_c/t) and residual tensile strength. Also, the relation between observed and predicted residual tensile strength considering pit characteristics are shown in Fig. 6. Determination coefficient of the model, considering pitting depth shows 0.5978.



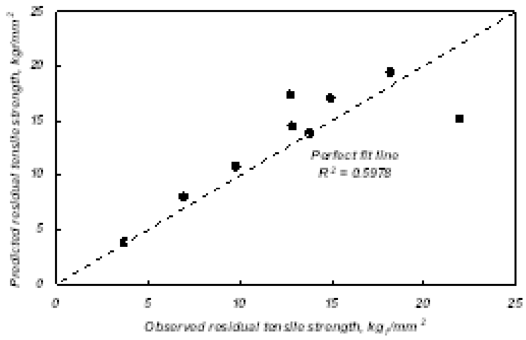
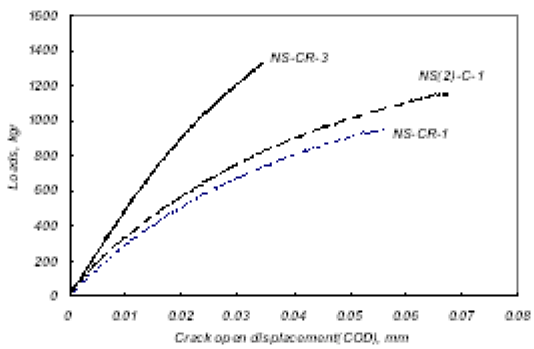


Figure 5 Pit characteristics (p_c/t) versus strength considering pit characteristics.

Figure 6 Observed and predicted residual tensile residual tensile strength.

For the prediction of residual strength by using fracture toughness, relation is analyzed between load and COD(Crack Opening Displacement) for each specimen. The pest was performed using DENT(Double-edge notch tensile) specimen, in simple tension test(Rajani et al., 2000).

Relation between load and COD(Crack Opening Displacement) and the results of toughness test can be seen in Fig. 7 and Fig. 8, respectively.



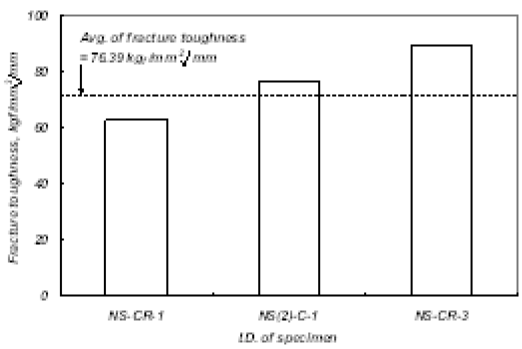


Figure 7 Load and crack opening displacement.

Figure 8 Results of toughness test.

In Fig. 9 Prediction results of residual tensile strength can be seen by fracture toughness and the model of fracture toughness is used which proposed by Rajani et al.(2000). The relationship between observed and predicted residual tensile strength considering fracture toughness are proposed in Fig. 10. Determination coefficient of the model, considering fracture toughness shows 0.62.

The proposed models, considering tensile strength and fracture toughness are expected to become helpful tools to estimate residual life of water pipe in the planning of rehabilitation/replacement.

Conclusions

The goal of this study is to assess cast iron pipes (CIPs) and present a residual tensile strength prediction model using pit characteristics and fracture toughness.

It is shown that an average pit depths of collected CIPs were in the range from 0.63 to 6.49 mm, loss of tensile strength compared with net metallic tensile strength were from –7% to 68%. Also it is expressed that the fracture toughness for NS-CR-1, NS-CR-2, and NS(2)-CR-1 were in the range from 62.85 to 89.39 kgf/ mm^2, and average of those samples was 73.69 kgf/mm^2 on CIPs.

The proposed models in this study by using pit characteristics and fracture toughness showed a good correlation for measured residual tensile strength.

Acknowledgement

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References

Rajni B., Maker J.(2000), "Investigation of grey cast iron water mains to develop a methodology for estimating service life", *AWWARF*, pp.1~249.

Seica, M.V., Packer, J.A., Grabinsky, M.W.F., Adams, B.J., Karney, B.W., "Evaluation and testing of cast iron and ductile iron water main samples", Final report to city of Toronto, Department of civil engineering, University of Toronto, Ontario, pp.1~203.

Seica M.V., Packer J.A.(2004), "Mechanical properties and strength of aged cast iron water mains", *Journal of materials in civil engineering*, Vol.16, No.1, pp. 69~77.

Atkinson K., Whiter J.T., Smith P.A., Mulheron M.(2002), "Failure of small diameter cast iron pipes", *Urban Water*, pp.263~271.

Deb A.K., Grablitz F.M., Hasit Y.J., Snyder J.K., "Prioritization water main replacement and rehabilitation", *AWWARF*, 2002. pp.1~113.

Smith, R.A.(1977), "A simplified method of predicting the rates of growth of cracks initiated at notches", *Fracture mechanics in engineering practice*, P. Stanley, Ed. Applied Science, London, pp.173~182.

ASTM(1996a), *Standard testing methods for tension testing of metallic materials*, ASTM E8-96a. New York: ASTM.

ASTM E8-96a(1996). *Standard testing methods for tension testing of metallic materials*, New York: ASTM.

ASTM E399(2005), *Standard test method for Linear-elastic plane-strain fracture Toughness* K_{IC} *of metallic materials*, New York: ASTM.

Snap, Crack, Pop - Recording of a Prestressed Pipe Failure

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Abstract

The San Diego County Water Authority has over 400 kilometers of large diameter pipelines that provides up to 95 percent of the water to San Diego County's 3 million people. Of the 400 km of pipelines, 133 km is made of prestressed concrete cylinder pipe (PCCP) manufactured between 1958 and 1982, and ranging in diameters of 168 cm to 244 cm. In order to provide a safe and reliable supply of water to San Diego County, the Water Authority implemented the Aqueduct Protection Program (APP) in 1992 which includes the condition assessment of the Water Authority's PCCP. Part of the program consists of developing and researching new non-invasive/non-destructive inspection procedures to determine the condition of existing PCCP.

In March 2006, the Water Authority installed 17.1 km of acoustic fiber optic (AFO) cable to monitor wire break activity and to provide critical information on the remaining service life of the pipeline. During the commissioning of the AFO system, there was a catastrophic failure on a section of the pipeline. This paper discusses the equipment, results and accuracy of the AFO system.

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Introduction

The Water Authority has an aggressive internal inspection and pipeline condition assessment program that began in 1992. This program is part of the Water Authority's Aqueduct Protection Program (APP), which was initiated to ensure that the Water Authority's pipelines continue to deliver water without interruption. The APP has been successful in identifying distressed pipe sections which allows for proactive repair or replacement during scheduled pipeline shutdowns. These procedures have been applied to all of the pipeline materials but this paper will only address the APP as it applies to the Water Authority's Prestressed Concrete Cylinder Pipe (PCCP) inventory, and more specifically, some select sections of the Pipeline (Figure 1).

Figure 1 – Aerial Overview of Pipelines 3 and 4

In 1999, the Water Authority supplemented their internal inspection and sounding procedures with other Non Destructive Testing (NDT) technologies including eddy current electromagnetic inspection technologies. These technologies work by inducing a varying electrical field in the prestressing wires and measuring the corresponding magnetic field response. The main components of the equipment include a transmitter coil to generate the field, a receiver coil to pick up the induced response, and a data logger to record the data. This equipment is moved through the pipeline and the resulting data is recorded on the data acquisition system. By evaluating this data, wire breaks can be identified on pipe sections and the position and number of wire breaks can be estimated.

These technologies generally provided improved information on the location and the number of broken prestressing wires in each pipe section. The results from the electromagnetic inspection provide an indirect assessment of the pipeline and the baseline condition of the pipeline can be established. This information is useful, but does not provide information about the ongoing rate of deterioration of the pipeline. To establish the rate of deterioration, acoustic monitoring can be implemented to record the time and interval of ongoing wire breaks in the pipeline without taking the pipeline out of service.

Acoustic monitoring is a 'passive' technology that is often installed while a pipeline is under normal operations. The technology relies on the water in the pipe as the medium to transmit the acoustic wave, generated by a wire break, to each of the sensors. Once detected, the arrival time of the 'wire break signal' to each of the sensors is calculated and the location of the event determined. Acoustic technology can be used for both short-term monitoring (3-6 months) and long-term monitoring (many years).

In 2005, as part of the ongoing evolution of the Water Authority's APP, an acoustic monitoring program was implemented and 6 hydrophone arrays where deployed over approximately 7.3 km of pipeline. A total of 19 wire breaks where recorded during the 12 month monitoring program. The results of the acoustic monitoring program corroborated the electromagnetic inspection information and provided the Water Authority with improved information to plan and schedule repairs with increased reliability.

While monitoring one of the hydrophone arrays, a number of acoustic events were recorded that were unlike wire breaks but also very unusual for an operating pipeline. These events where reported to the Water Authority and an employee was dispatched to the location to investigate the source of the events.

Upon arrival to the site, a contractor was found with an open excavation directly over the Water Authority's 96-inch pipeline. At the time, the pipeline was operating at 200 psi. Fortunately, the excavation was backfilled and a potential catastrophic failure was averted. This was the first time acoustic monitoring had been used successfully to identify third party damage. As a result, the system is now being considered as part of a 'Right of Way' management program focused on reducing the incidence of third party or contractor related damage to the pipeline.

The initial success of the acoustic monitoring program in 2005 convinced the Water Authority to continue the acoustic monitoring program into subsequent years. In 2006, the Water Authority advertised two separate requests for proposals that included electromagnetic inspection and acoustic monitoring. The combination of these inspection and monitoring techniques provides increased confidence in the pipelines condition allowing for the implementation of more effective management strategies.

2006 Pipeline Inspection Program

In 2006 a contract was awarded to two firms; the first, to perform eddy current inspection on 24 km of Pipeline 3 and 9 km on Pipeline 4. The second included acoustic monitoring of approximately 7.3 km of portions of the same pipelines.

Pure Technologies (Pure) was retained by the Water Authority to deploy an acoustic monitoring system over approximately 7.3 km of the pipeline. The initial plan was to utilize hydrophone arrays (1.6 km in length) with up to 32 hydrophones connected to the array cable. The hydrophone array is typically deployed into the pipeline while the pipeline is under pressure and normal operation (Figure 2). Once deployed, the cable is connected to a local data acquisition system which performs continuous hardware and software filtering of the recorded acoustic activity. When a wire break occurs, the event is stored locally and sent over the Internet to a data processing facility for review and classification.

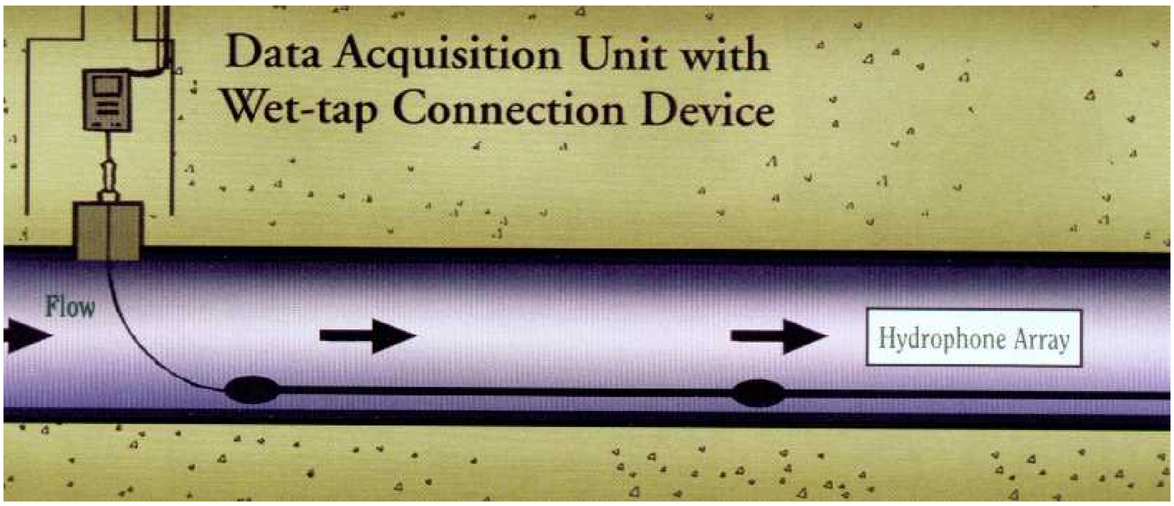

Figure 2 – Schematic of Hydrophone Array Installation

Between the 2005 and 2006, Pure was finalizing the development of an Acoustic Fiber Optic (AFO) monitoring system. The AFO system has a number of advantages over existing hydrophone and other sensor technologies, including:

- The ability to monitor up to 19 km of fiber optic cable from one computer.
- The fiber optic sensor is continuous over the entire length of cable
- The fiber is ideally suited to ‘permanent or long term’ monitoring
- Reduced cost when compared to other existing acoustic technologies
- The fiber could be utilized for other applications including; high bandwidth communication, third party damage and possibly as a permanent leak detection sensor

The original scope of work, as defined in the Water Authority's contract documents, included three months monitoring of approximately 7.3 km of pipeline. Pure Technologies believed extending the fiber optic sensor would provide the Water Authority with increased flexibility in the future management of the pipeline while reducing the need for dewatering and subsequent inconvenience to the member agencies. While the pipeline was dewatered, Pure Technologies installed fiber optic cable in approximately 17.1 km of pipeline. Installation of the systems took place in March 2006, during a scheduled shutdown and internal inspection of the prestressed concrete cylinder pipe (PCCP).

Acoustic Fiber Optic (AFO) System

The AFO system is comprised of a specially designed fiber optic cable that is deployed inside a pipeline. The cable is attached to the invert of the pipe at regular intervals using rubber clamps and expandable hoops (Figure 3 & 4). Since cable movement affects the acoustic capabilities of the system, the cable should be secured at points of inflection and in areas of turbulence in the pipeline.

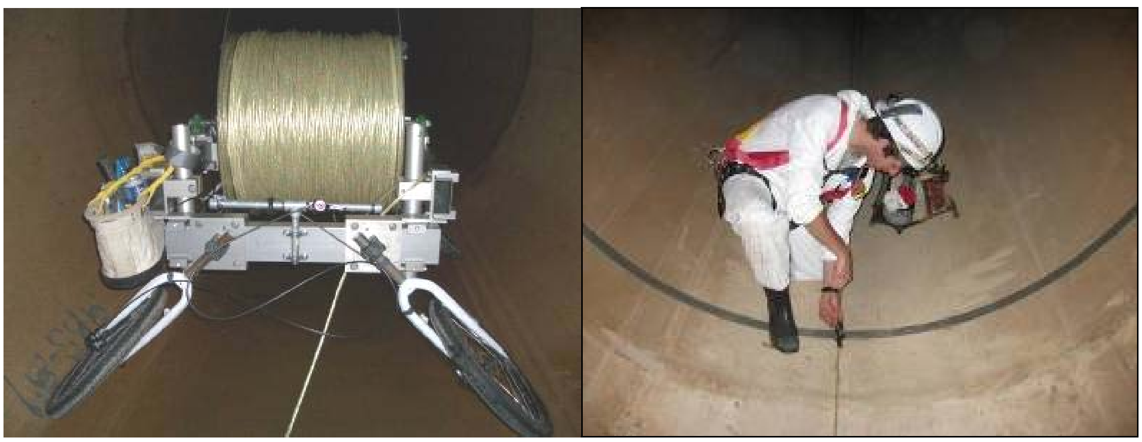

Figure 3 & 4 – Fiber Optic Cable Installation

The fiber exits and enters the pipeline periodically through a combination of taps and specialty valves called wipers. Junction boxes are installed at the cable exit points to facilitate splicing of the cable and intermediate test points for future maintenance if needed.

As per standard acoustic monitoring systems, the fiber sensor is attached to a Data Acquisition (DAQ) system that is continuously acquiring acoustic data from the fiber (Figure 5). The DAQ processes the data and rejects any that are outside of the parameters established for an event of interest (e.g., wire break). Conventional telephone lines, wireless or wired internet connections transmit the remaining data to a central processing facility.

Figure 5 – Optical Data Acquisition System

Once the data has been received, data processors carefully analyze each event. Acoustic events that meet the criteria for a wire break are classified as such and accurately located.

Email notifications are automatically sent to client personnel as soon as the event has been located and classified by the processing team. This provides instant access to the information, and also a notice to proceed to a web-based management site where more detail on the event can be determined.

Histograms are generated for each site showing the location of wire breaks. Acoustic events determined to be unrelated to pipe distress are not reported or located.

2006 Acoustic Monitoring Program

Two monitoring sites were established following the installation of the AFO in March 2006. The first site was configured to monitor one pipeline for approximately 6.7 km. The second system was configured to monitor another pipeline for approximately 10.4 km. Satellite communications were used to transfer data for the two monitoring systems. Following the commissioning of the sites, it became apparent that the satellite system used was insufficient as data transfer was limited and unacceptably slow.

During the commissioning of the AFO systems, a wire break was detected on Sunday May 7, 2006 with an approximate location. The Water Authority checked their database and found that the approximate location of the break was on a section of 168 cm PCCP that had two areas of predicted wire breaks from the electromagnetic inspections. The one additional wire break brought the total breaks to 16. The number of breaks combined with the relatively low pressure of 120 psi did not cause the concern to shut the pipeline down. Following the detection of two more wire break events on Sunday May 14, the Water Authority was notified on Monday May 15 of the locations of the additional two wire breaks. Again, with only 18 total wire breaks the pipe's structural integrity was not compromised to withstand the operating pressure.

During the evening of May 15, the section of pipe with the three wire breaks detected on May 14 ruptured. Data analyzed on Tuesday May 16 revealed eighteen additional wire breaks located in same approximate location further to the three reported the day before. While slow communications impeded data transmission and prevented the reporting of the last event from May 14, most of the wire breaks occurred after processing for the site took place on May 15. The following graph (Figure 6) shows the rapid increase in the rate of deterioration of the pipe section.

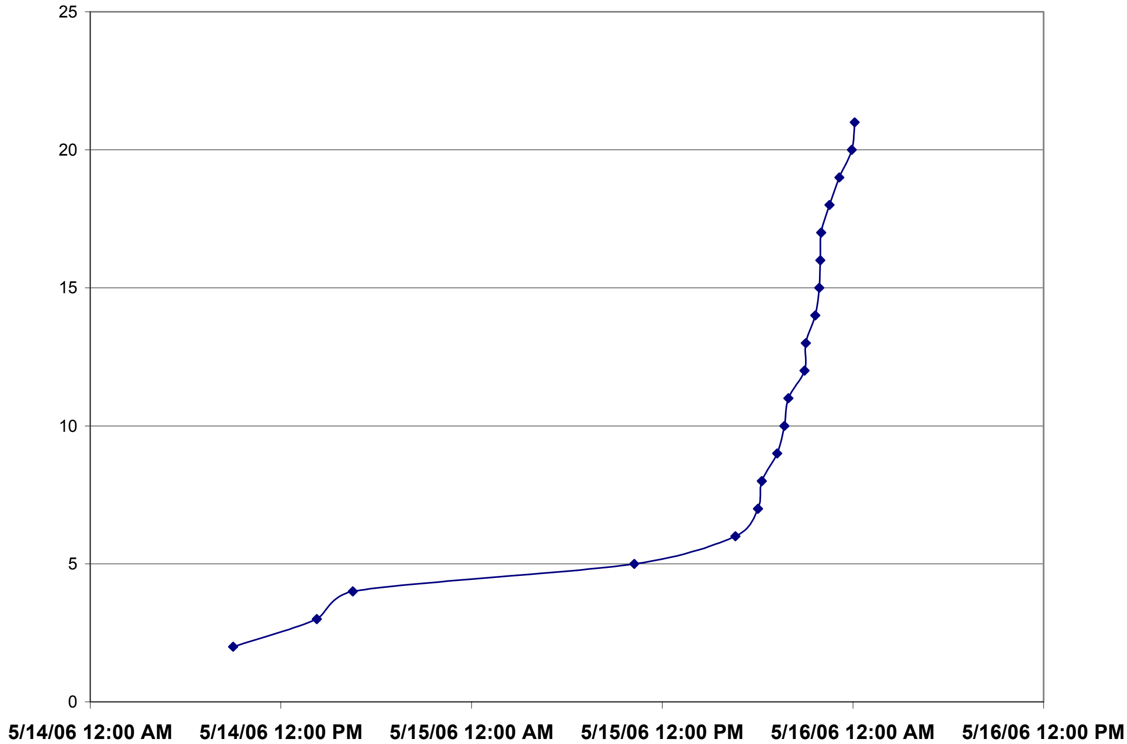

Figure 6 – Wire break activity leading up to the pipeline failure

The rupture resulted in the shutdown of the pipeline, and reconfiguring of the AFO monitoring system. Site 1 was decommissioned as the majority of the pipeline previously monitored by this site was left empty.

Due to the rapid progression of failure recorded on this pipeline, and in response to the Water Authority's request for more timely information from the monitoring systems, Pure Technologies has implemented additional reporting and processing protocols. While outside the typical monitoring requirements for pipelines, the pipeline rupture in May 2006 highlighted the importance of providing timely information to the Water Authority to assist in managing the pipeline.

Close to real time information on the time and locations of wire breaks occurring in a monitored pipeline is typically provided. While many acoustic events are detected by the monitoring system every day, very few pass through the hardware and software filtering on site.

Although the above described reporting protocol is usually adequate for most pipelines, the Water Authority requested a more immediate reporting protocol be established to avoid any future rupture of this critical pipeline. In response, the Water Authority has been given access to the onsite acquisition computer so data can be reviewed more frequently, or during times outside of regular business hours. The Water Authority is the first agency to have this type of access.

Software tools have been added to the onsite data acquisition system with a graphical representation of the acoustic events along with aerial images that provide information on the location of the event. These tools in combination with remote access allow Water Authority personnel real time analysis of their pipelines.

Conclusions

The May 2006 pipeline failure resulted in the longest and most complex emergency repair in the history of the Water Authority. After three emergency construction contracts, multiple professional service agreements and many staff hours, all emergency work associated with failure was completed on October 29, 2006. The information collected from the forensic investigations and the acoustic fiber optic monitoring system has initiated the re-evaluation of the rehabilitation/replacement schedule for the Water Authority's PCCP. Part of this evaluation includes the use of alternative monitoring such as acoustic fiber optic technology.

AFO technology has worked well for the Water Authority and has provided an additional tool in the condition assessment tool bag to manage the buried asset. This success wasn't without challenges though. The change in elevation and the extremely steep slopes in San Diego proved to be a challenge when securing the cable inside the pipe. With assistance and experience from the Water Authority's staff, the contractor was able to install and secure cable.

The power of water inside the pipeline is generally overlooked. One of the valuable lessons learned was how the velocity of the water and open channel flow can affect the AFO system's ability to record and detect the location of wire break events. The combination of open channel flow coupled with increased water velocity, in excess of 20 feet per second, resulted in excessive background noise that was difficult to manage with both software and hardware filtering.

The Water Authority's Aqueduct Protection Program continues to evolve relying on a combination of sound engineering judgment coupled with advanced condition assessment and monitoring technologies. The success of the acoustic fiber optic monitoring system has provided the Water Authority with improved ways to manage long lengths of pipe with limited impact on water delivery. The information provided by long term acoustic monitoring will be used to extend the service life of these pipelines while averting the costs associated with emergency repairs resulting from catastrophic failures.

Failure of Prestressed Concrete Cylinder Pipe

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Abstract

Prestressed concrete cylinder pipe (PCCP) tends to be of large diameter, making many failures of this type of pipe relatively catastrophic and costly. To date, most research has been focused on PCCP inspection technologies and performance prediction in order to minimize the risk to utilities from failures. The performance of Prestressed Concrete Cylinder Pipe has been an item of interest to water utilities for a long time. Fifteen years ago, AWWA Research Foundation (AwwaRF) and the Bureau of Reclamation initiated a study, "Performance of Prestressed Concrete Pipe" that was unfortunately never published. AwwaRF has also funded studies to develop software that uses fuzzy logic (Kleiner, et al. 2005) to predict future PCCP behavior useful if a utility has sufficient data from actual field experience with PCCP. Another AwwaRF report (Romer, et al. 2005), developed methodologies, techniques, protocols, and technology for identifying the specific conditions that lead to corrosion failures of PCCP. That study included testing the effects of cathodic protection on pre-existing notches in PCCP reinforcing wires, and compared the accuracy of similar internal investigative tools for the detection of broken PCCP reinforcing wires.

This paper presents preliminary results of the most recent AwwaRF funded study, intended to collect, codify, expand upon, and disseminate the results of the previous studies on PCCP failures. The project began with a survey of a selected group of water

utilities with a fair amount of PCCP in their systems, selected carefully to represent the range of conditions and variables that affect PCCP performance across North America. The survey results, combined with a pre-existing database of over 450 PCCP failures, were statistically projected over the installed PCCP base of North America, and provide a surprising indicator of failures and failure rate of PCCP.

Introduction

The report of water pipelines failing are a fairly commonplace news item, particularly where the resulting flood damage provides spectacular footage for the 10 o'clock news. These failures, some of which are PCCP, also cost significant sums for repair and eventual pipeline replacement. These failures can wash out parallel sanitary sewers, which present a public health problem due to the possible contamination of the drinking water supply, and can also destroy private property (Henry 2005 and Ortega 2005). Corroded or damaged PCCP pipelines have explosively failed during rapid changes in pipeline operation, such as during high demand periods, fire suppression, and in the middle of heat waves.

The performance of PCCP has been an item of interest to water utilities for a long time. Fifteen years ago, AwwaRF and the Bureau of Reclamation initiated a study entitled "Performance of Prestressed Concrete Pipe" that was unfortunately never published. This study is intended to disseminate some of the answers that were to be provided by that study.

This project assembled and presented data to provide an understanding of the trends of the number of failures and the failure rate of PCCP in North America over the past 20 years. A preliminary database of 425 geographically diverse PCCP failures was updated through 1999, with the data through 2006 to be presented in another report. A survey and workshop was conducted, including water utilities with a fair amount of PCCP in their systems. The objective is to make this a utility/user-driven project, utilizing the following approach:

- Define and find the failures, however and wherever possible.
- Solicit, collect, and review all possible information about PCCP design, operation, and failures.
- Scrutinize and summarize what is "known."
- Understand and state what is "not known."
- Report what was concluded by others.
- Take a critical look at the conclusions in light of the increasing knowledge base.
- Use statistical tests to determine important factors and begin predictive modeling.

Please note that cited failure rates of PCCP are intended to be per mile per year (occurrences per mile per year). It will be really interesting to calculate this for lined cylinder pipe (LC-PCCP) and embedded cylinder pipe (EC-PCCP), Class III vs. Class IV, all other Classes vs. Classes III and IV, Interpace vs. all others, but normalizing factors need to be developed that relate the probability (per mile per year). These data are what was obtained from the users. This approach is similar to what the nuclear people do when they quote likelihood per reactor year (i.e., four per reactor year; if you have 100 reactors that operate for 100 years, you will have one failure). Remember, one of the things that has to be defined early on is what constitutes a failure. Is a leak a failure? Are broken wires above the critical unsupported length factor (Romer, Bell, et al. 2004) failures? This too requires utility input.

History of PCCP

Early reinforced concrete pressure pipe was an extension of concrete culvert pipe, with addition of a steel cylinder for water retention. Spirally wound reinforcement has been reported as early as 1929 (Lynch 2004) for concrete pressure pipe. In response to the high demand for steel in the war years, the emerging technology of prestressing concrete was applied to water pipelines. The earliest application in the U.S. was in 1942 (AWWA C301-99 Foreword). That pipe, which is now known as lined-cylinder type PCCP (LC-PCCP) consists of a steel cylinder with cast concrete core, over-wrapped with steel wire. Continuing scarcity of steel, coupled with successful application to smaller pipe, led to the introduction of embedded-cylinder type PCCP (EC-PCCP) in 1953 (AWWA C301-99 Foreword). Site-manufactured EC-PCCP has been constructed as large as 256 inches in diameter.

Significant differences in the manufacture of LC-PCCP and EC-PCCP not only exist, but the configuration of each type has changed significantly throughout the years of manufacture. Table 1 lists some of the major differences in the pipe, and Table 2 lists some of the significant changes in the products.

Definition of PCCP Failure

Similar to the human population, each individual “more or less 16-foot” pipe section has a birth (manufacturing) to death cycle that is affected by its heredity (design, manufacturer, materials, etc.), birth defects (construction, installation), and lifestyle (operations and maintenance, etc.). For the purposes of this study, failure has been defined as the loss of use of a pipe section or reduction in confidence in that pipe section to continue in service, after discovery of a pipe section deficiency. This includes repair, replacement, or reduction in operating pressure. Three categories of PCCP failure have been defined:

1. Catastrophic ruptures (Category I)
2. Failure discerned by inspection (Category II)
    - Visual, sounding, and accidental discovery
    - Electronic inspections
3. Loss of service (Category III)
    - Time out of service
    - Full or partial replacement

Definition of Expected Service Life for PCCP

We all know that as you get older, you get closer to death. The same is true for pipe, it's just that the life expectancy varies by many factors. Life expectancy varies from 50 years, to 100 years, to "indefinite" depending on the perception of the pipeline owner.

Common Causes of PCCP Failure

Causes of failure of PCCP are numerous. They include high chloride environment (Villalobos 1998), the quality of the mortar including lack of complete envelopment of the prestressing wires within the cement mortar coating (Price 1998), the reinforcing wires (Walsh 1998 and Knowles 1990) corrosive soils (Galleher 1998), inadequate thrust restraint (Ojdrovic 2001), construction damage (Parks 2001), cracks in the cylinder welds (Price 1990), delamination of the coating (Price 1990). Details of the failure mechanism of EC-PCCP with broken prestressing wires have been published (Zarghamee 2001). That detail may be of use only in the courtroom, however, because once the pipe condition has approached the imminent failure state, it is too late.

Research to Identify PCCP with High Likelihood of Failure

Failures of PCCP pipelines prompted visual inspections. Those inspections often revealed cracks were on the interior, leading to further investigations. Interior cracks sometimes indicated disbondment of the concrete core from the steel cylinder, and it was soon discovered that tapping on the interior (called "sounding") was an effective means for identifying those areas. Internal inspections have been supplemented with ultrasonic examination to infer the condition of the core (Lewis 2005). An AwwaRF study (Jackson 1992) reviewed the available nondestructive evaluation technology for waterlines, some of which are applicable to PCCP. But these do not provide any significant indication of the structural integrity of the prestressing wires.

An alternative inspection methodology was made commercially available in 1997, by utilizing the wires within the pipe as a radio-frequency measurable coil antenna (Mergalas 2001b). Because that technique promised to identify the number and location of wire breaks in each pipe length inspected, the PCCP owning utilities were interested in its development to the extent that AwwaRF funded a study (Mergalas 2001a).

Recently, acoustic monitoring of in-service EC-PCCP pipelines has been utilized to identify actively breaking wires (Diaz 2005 and Worthington 1996). Another real-time technique reported to be successful is Inductive Scan Imaging (Almugherty 2005), a technique that to date has been used only on the exterior of PCCP. Methods developed to determine the number of wire breaks in EC-PCCP have recently been extended to LC-PCCP (Xianjic 2005 and Mergalas 2005). These methods have widely been promoted to determine the condition of PCCP pipelines, yet their accuracy has not been as yet demonstrated sufficient to rely entirely thereon (Galleher 2005, Bambei 2005, and Parks 2001). Water utilities, desperate for concrete answers, continue to fund research (Bengtsson 2005).

It is clear that the time to more actively manage the remaining service life of PCCP pipelines is at hand. Evaluation and management of PCCP pipelines by utilities has been approached on the basis of evaluating existing information (Bichler 2005), and a risk-based approach has been presented (Romer 2004). A typical PCCP pipeline assessment approach begins with data collection and analysis (Bell 2001):

- Data collection, design and shop drawings.
- Complete system surge analysis.
- System operation modifications.
- Corrosion survey and alignment corrosivity analysis.
- Nondestructive investigations.
- Detailed structural integrity evaluation.

Crucial data for analyses are often lacking, even at a utility with comprehensive records (Galleher 2001). Models have been developed to estimate failure risk using fuzzy Markov techniques (Marshall 2005 and Kleiner 2004), when data are scarce or unreliable. It is not clear that the additional effort provides any greater level of confidence to the utility, because insufficient data generally exist.

Perhaps the most effective, on an individual pipe basis, is analysis of the structural integrity of the pipe, using the best available data. Those analyses can be as complicated as finite-element models (Lofti 2005 and Diab 2001) or analysis using the best-available analytic methods at the time the pipeline in question was originally designed (Lewis 2005). Some finite-element models possess a complexity far in excess of the original pipe design (Gomez 2004).

Alleviating PCCP Failures

Initial steps at risk management included application of cathodic protection to the prestressing wires (Zarghamee 1998), assuming that cathodic protection could reduce the rate of corrosion for corrosion-based failures. Because of the high numbers of PCCP, pipelines manufactured by Interpace and with Class IV wire, one utility used linear extrapolation to estimate time of failure (Bradish 1995).

Remote field-eddy current inspection, a technique in use by the city of Calgary, was initially reviewed in an AwwaRF study (Jackson, et al. 1992). This type of inspection, however, is only a component of risk management (Mergalas 2001c).

Uncontrolled or unidentified stray currents can cause rapid deterioration of PCCP pipelines due to Hydrogen Embrittlement (HE). Even cathodic protection as applied to PCCP in an effort to control corrosion has been attributed as the cause of failures (Marshall 1998). Methodologies to identify sources of and isolation of pipelines from stray currents and electric utility grounding issues were identified in two AwwaRF studies (Duranceau, et al. 1996 and Romer, et al. 2004). Others have proposed pulsed cathodic protection (Doniguian 1998).

AwwaRF has funded a workshop (Lillie 2004), which summarized the devices available to utilities for inspection of water transmission mains. In that study, the economics of condition assessment were evaluated against the deferral of capital (replacement of the pipeline) and operating expenses (repair/rehabilitation). An equally recent AwwaRF study (Reed 2004) addressed means of continuous monitoring of the structural capacity of transmission mains. Clearly, there is a significant interest in the preservation and the extension of useful life of water mains and, in particular, PCCP. Utilities have taken steps in advance of failure to reline (Fiori 2001), utilizing carbon fiber (Moncrief 2001) and steel cylinders (Suydam 2001) and replacement with new steel pipe subsequent to failure.

Significance to Water Utilities

PCCP users know that they have a potential problem. Every pipe has a story to tell. Central to the cause of problems in waterworks is incomplete data, which would otherwise allow rational decision-making. The problem is that the data are dispersed and diffuse. Thus, a primary goal of this study is to collect the data and put it into place at one time. The key is "mining" the data from the pipeline "community." Once all the data are in hand, the real project is to turn the data into useable information, in the form of graphics and decision tools.

Cost of Failures

An accurate method of estimating the tangible and intangible cost of transmission and distribution water mains failures was introduced in an AwwaRF study (Cromwell 2002). In that study, electric, transportation, natural gas, and emergency planning industry methods for estimating "customer outage costs" were extrapolated to the waterworks industry.

The cost of replacement of the nation's inventory of PCCP has been estimated at over $40 billion (Megalas 1998). The cost of maintenance of a PCCP pipeline can also be excessive (Marshall 2001). AwwaRF has funded studies that focus on the rehabilitation of distribution mains (Deb 1990 and O'Day 1984) wherein the cost of rehabilitation of small-diameter mains approaches the cost of new pipelines. The economics have not changed in the 16 years since publication of the most recent of those reports.

Relining PCCP in-place has been completed with steel cylinders, starting with the Bureau of Reclamation's Jordan Aqueduct in 1984. Many large aqueducts have been rehabilitated, at great cost, in this manner (Stine 1998 and Khondiker 1998) with sometimes-significant reduction of flow capacity.

Survey of Utilities to Supplement Existing Database

Probably more important than simply identifying PCCP leaks and breaks was to establish if there is a pattern of leaks and breaks in PCCP. The Water:\Stats database contains a tremendous amount of data, but because it lacks information regarding the age, wire type, and manufacturing characteristics of PCCP, it is not very useful for this study. A more useful set of data was compiled for AwwaRF 15 years ago and is summarized in

Tables 3 and 4. The PCCP failure rate at that time was not significantly greater than other pipeline materials, based upon survey results from 114 utilities.

Figures 1 and 2 depict data collected since that unpublished study, including information on 435 PCCP failures, but omit the cases where data are missing or incomplete. These data represent the most complete body of PCCP failures to the year 2000 extant. Additional and much more recent data have been collected and are being checked for future inclusion in this database.

The likelihood of PCCP failures can be inferred from Figures 3 and 4. It appears that the number and rate of LC-PCCP failures has dramatically been reduced since 1984. The cause of that reduction may be due to aggressive action on the part of the utilities to identify the pipe with the highest potential for failure in their systems and take action. The reduction in total number of failures may also be attributed to other factors still being evaluated. Statistical analysis of data to confirm hypotheses will be published later. The incidence of reported EC-PCCP failures to 2000 similarly appears in Figure 4 to be increasing; however, the aggressive action on the part of the utilities to identify the pipe with the highest potential for failure in their systems may also be the apparent "cause." That is because the data are skewed towards wire breaks detected by NDT, each of which is categorized as a failure. There is no dispute with that inference, yet the magnitude of the failures thus categorized is less critical than ruptures.

Interestingly, the data suggest that the likelihood of failure decreases with age. This implies a preponderance of reported failures due to manufacturing or installation related deficiencies, which occur early in the life of the pipe.

Predicting Failures

The determination of the remaining useful life of pipelines, not just PCCP pipelines, has been reported utilizing probabilistic or statistical methods to estimate a survival function on the basis of past behavior (Nelson 2005). Assignment of a pipe criticality index included web-based data in a very large PCCP system (Essamin 2005) for EC-PCCP. Artificial Neural Networks (Najafi 2005) have been utilized to predict behavior of systems based upon past behavior. The remaining service life has even been estimated on its actuarial value (Baik 2004), although it is understood that utilities would like to believe that their underground assets have an indefinite useful life. The mathematics proposed (Kleiner 2005) for managing the risk inherent with PCCP pipes may be significantly daunting to management, whose eyes roll at the thought of manipulating fuzzy-based Markov techniques, even when aided by computer programs. What is necessary is a risk assessment system whose results are readily understandable by water utility management.

A timeline has been prepared to allow comparison of changes in manufacturing and design standards for PCCP as well as changes to the components (e.g., wire). Production data from 10 manufacturers representing 100 million feet of PCCP along with pipe failure data have been included on the timeline. Changes to the AWWA standard have been numerous and sometimes significant. For example, AWWA 7B.2-T (1949 tentative standard) required "...the steel cylinder and wire shall reach their respective elastic limits simultaneously at a pressure equivalent to at least 2 1/4 times the normal water pressure.…" AWWA 301-58 required "At a pressure equal to twice the design pressure, the stress shall not exceed its original gross wrapping stress."

Preliminary examination of the timeline reveals that the pipe manufactured subsequent to issue of the 1964 edition of AWWA C301 (and prior to the 1992 standard) may be of greatest risk of premature failure. In that 1964 standard, the design procedures A and B were adopted...moving the design criteria from the body of the standard to an appendix. Also in that standard, there was an 18.5 percent reduction in minimum wire size, a 37.5 percent increase in concrete core stress at time wire is wrapped, a 16.7 percent reduction in the minimum amount of portland cement in the core, a 20 percent reduction in the minimum coating thickness, a six-fold increase in the joint tolerance, and there was still no minimum cylinder thickness (since 1955 edition). The availability of "Class III" wire beginning in 1968 and the availability of "Class IV" wire between about 1972 to 1979 may have contributed to the increased number of reported failures (Brandish 1995).

The statistical relevance to PCCP life due to improvements to the standards is still being investigated. We have answered our own question (and that of the industry)...what was the most important aspect to know? It's the wire stupid...that is because three things (mistakes) happened simultaneously: One, two, three strikes and you're out.

1. Increase in production of PCCP...quality always goes down,
2. Change in design standards...to the less conservative side, and
3. Unregulated wire changes...(Classes III and IV).

It appears that that will tell most of the tale...year of manufacture/installation, design standard, and wire type.

References

Almughery, A., et al. (2005), *Inductive Scan Imaging Technique in Detecting and Visualizing Corrosion for Large-Scale Prestressed Concrete Cylinder Pipes*, Pipelines 2005, ASCE, Reston, VA, pp. 980-988.

ANSI/AWWA C301-06 (Draft), *Standard for Prestressed Concrete Pressure Pipe, Steel Cylinder Type*, American Water Works Association, Denver, CO, Foreword.

Baik Shik, Hyeon, et al. (2004), *Impacts of Deterioration Based Life Cycle Cost Analysis (LCCA) on the Assessment of Values of Wastewater Infrastructure Assets*, Pipeline Engineering and Construction – What's on the Horizon? John J. Galleher and Michael Stift, eds., ASCE, Reston, VA (CD-ROM).

Bambei, John H., Jr., et al. (2005), *Correlation of Wire Breaks on Prestressed Concrete Cylinder Pipe with Predictions from Electromagnetic Testing*, Pipelines 2005, ASCE, Reston, VA, pp. 989-1000.

Bell, Graham E. C., et al. (2001), *An Assessment and Reliability Program for PCCP*, Pipelines 2001-Advances in Pipeline Engineering & Construction, Joseph Castronovo, ed., ASCE, Reston, VA (CD-ROM).

Bengtsson, Brad, et al. (2005), *Next Steps in the Evolution of an Acoustic Emission Testing System*, Pipelines 2005, ASCE, Reston, VA, pp. 1025-1034.

Bichler, Robert (2005), *Evaluation and Management of Prestressed Concrete Cylinder Pipe Assets*, Pipelines 2005, ASCE, Reston, VA, pp. 116-129.

Bradish, Bryan M., et al. (1995), *When Should PCCP with Interpace Class IV Wire Be Replaced?*, Advances in Underground Pipeline Engineering, J. Jeyapalan, ed., ASCE, NY, NY, pp. 156-167.

Cromwell, John E., III, et al. (2002), *Costs of Infrastructure Failure*, Awwa Research Foundation (AwwaRF), Denver, CO.

Deb, Arun K., et al. (1990), *Assessment of Existing and Developing Water Main Rehabilitation Practices*, AwwaRF Denver, CO.

Diab, Youssef G. and Thomas Bonierbale (2001), *A Numerical Modeling and a Proposal for Rehabilitation of PCCPs*, Pipelines 2001-Advances in Pipeline Engineering & Construction, Joseph Castronovo, ed., ASCE, Reston, VA (CD-ROM).

Diaz, Robert, et al. (2005), *Acoustic Monitoring and Replacement of a Distressed 42-Inch Prestressed Concrete Transmission Main*, Pipelines 2005, ASCE, Reston, VA, pp. 390-400.

Doniguian, Ted, et al. (1998), *Extending the Life of Prestressed Concrete Cylinder Pipe With Pulse Cathodic Protection*, Pipelines in the Constructed Environment, ASCE, Reston, VA, pp. 367-376.

Duranceau, Steven J., et al. (1996), *Effects of Electrical Grounding on Pipe Integrity and Shock Hazard*, AwwaRF, Denver, CO, ISBN 0-89867-861.

Essamin, Omar, et al. (2005), *Risk Management System for Prestressed Concrete Cylinder Pipeline: Practical Results and Experience on the Great Man Made River*, Pipelines 2005, ASCE, Reston, VA, pp. 241-251.

Fiori, Steven R., et al. (2001), *Rehabilitation of Prestressed Concrete Cylinder Pipe – A Utility Perspective*, Pipelines 2001-Advances in Pipeline Engineering & Construction, Joseph Castronovo, ed., ASCE, Reston, VA (CD-ROM).

Galleher, John J., et al. (2005), *Comparison of Two Electromagnetic Techniques to Determine the Physical Condition of PCCP*, Pipelines 2005, ASCE, Reston, VA, pp. 401- 410.

Galleher, John J., Jr., et al. (2001), *Advances in Construction Documentation for Future Pipeline Condition Assessments*, Pipelines 2001-Advances in Pipeline Engineering & Construction, Joseph Castronovo, ed., ASCE, Reston, VA (CD-ROM).

Galleher, John J. and Michael T. Stift (1998), *Internal Inspection and Database Development of PCCP*, Pipelines in the Constructed Environment, ASCE, Reston, VA, pp. 721-730.

Gomez, Roberto, et al. (2004), *Structural Model for Stress Evaluation of Prestressed Concrete Pipes of the Cutzamala System*, Pipeline Engineering and Construction – What's on the Horizon? John J. Galleher and Michael Stift, eds., ASCE, Reston, VA (CD-ROM).

Henry, Gregory J., et al. (2005), *Fast-Track Repair of 42-Inch PCCP Water Main*, Houston, Texas, Pipelines 2005, ASCE, Reston, VA, pp. 161-171.

Jackson, Rodney Z., et al. (1992), *Nondestructive Testing of Water Mains for Physical Integrity*, AwwaRF, Denver, CO, ISBN 0-89867-620-7.

Khondiker, Sufian A. and John R. Mitchell, Jr. (1998), *Design and Construction Aspects of the PCCP Water Main Rehabilitation*, Pipelines in the Constructed Environment, ASCE, Reston, VA, pp. 612-621.

Kleiner, Yehuda, et al. (2005), *Risk Management of Large-Diameter Water Transmission Mains*, AwwaRF, Denver, CO, ISBN 1-58321-400-03.

Kleiner, Yehuda, et al. (2004), *Modeling Failure Risk in Buried Pipes Using Fuzzy Markov Deterioration Process*, Pipeline Engineering and Construction – What's on the Horizon? John J. Galleher and Michael Stift, eds., ASCE, Reston, VA (CD-ROM).

Knowles, William L. C. (1990), *Failure of Prestressed Concrete Embedded Cylinder Pipe, Design and Installation*, Kenneth Kienow, ed., ASCE, NY, NY, pp. 434-441.

Lewis, Richard A. and Paul Fisk (2005), *Detection of Prestressed Concrete Cylinder Pipe Thinning From Hydrogen Sulfide Deterioration*, Pipelines 2005, C. Vipulanandan and R. Ortega, eds., ASCE, Reston, VA, pp. 965-979.

Lillie, Kevin, et al. (2004), *Workshop on Condition Assessment Inspection Devices for Water Transmission Mains*, AwwaRF Denver, CO.

Lofti, Hamid R., et al. (2005), *Reliability Assessment of Distressed Prestressed Concrete Cylinder Pipe*, Pipelines 2005, ASCE, Reston, VA, pp. 838-852.

Lynch, Brendan, et al. (2004), *Ten Times in Ten Years, Pipeline Engineering and Construction – What's on the Horizon?* John J. Galleher and Michael Stift, eds., ASCE, Renton, VA (CD-ROM).

Marshall, David H., et al. (2005), *Tarrant Regional Water District's Risk Management Plan for Prestressed Concrete Cylinder Pipe*, Pipelines 2005, ASCE, Reston, VA, pp. 853-861.

Marshall, David, et al. (2001), *Managing Maintenance Costs of a Water Transmission System*, Pipelines 2001-Advances in Pipeline Engineering & Construction, Joseph Castronovo, ed., ASCE, Reston, VA (CD-ROM).

Marshall, David H. (1998), *TRWD Experience with Prestressed Concrete Pipe*, Pipelines in the Constructed Environment, ASCE, Reston, VA, pp. 556-565.

Mergalas, Brian, et al. (2005), *Using a Combination of Condition Based Asset Management Techniques to Manage a Water Transmission System*, Pipelines 2005, ASCE, Reston, VA, pp. 782-792.

Mergalas, Brian and Xiangjie Kong (2001a), *Electromagnetic Inspection of Prestressed Concrete Pressure Pipe*, AwwaRF, Denver, CO.

Mergalas, Brian J., et al. (2001b), *NDE Inspection of PCCP Using Remote Field Eddy Current/Transformer Coupling*, Pipelines 2001-Advances in Pipeline Engineering & Construction, Joseph Castronovo, ed., ASCE, Reston, VA (CD-ROM).

Mergalas, Brian J., et al. (2001c), *Risk Management: A Case Study in Operating PCCP*, Pipelines 2001-Advances in Pipeline Engineering & Construction, Joseph Castronovo, ed., ASCE, Reston, VA (CD-ROM).

Mergalas, Brian J. and David L. Atherton (1998), *In-Line Electromagnetic Inspection of PCCP*, Pipelines in the Constructed Environment, ASCE, Reston, VA, pp. 714-720.

Moncreif, W. Jeffrey, et al. (2001), *Rehabilitation of 78-Inch PCCP with Carbon Fiber Reinforced Composite Material*, Pipelines 2001-Advances in Pipeline Engineering & Construction, Joseph Castronovo, ed., ASCE, Reston, VA (CD-ROM).

Najafi, Mohammad, et al. (2005), *Pipeline Condition Prediction Using Neural Network Models*, Pipelines 2005, ASCE, Reston, VA, pp. 767-781.

Nelson, Richard E. (2005), *Utilizing Condition Assessment Data in Pipeline Replacement and Reinvestment Planning*, Pipelines 2005, ASCE, Reston, VA, pp. 130-136.

O'Day, D. Kelly, et al. (1989), *Water Main Evaluation for Rehabilitation/Replacement*, AwwaRF, Denver, CO.

Ojdrovic, Rasko P., et al. (2001), *Condition Assessment of a PCCP Line Accessible from Outside Only*, Pipelines 2001-Advances in Pipeline Engineering & Construction, Joseph Castronovo, ed., ASCE, Reston, VA (CD-ROM).

Ortega, Rafael, et al. (2005), *Pipeline Ruptures: High Demand for Water Prompts an Innovative Solution*, Pipelines 2005, ASCE, Reston, VA, pp. 439-450.

Parks, Randall R., et al. (2001), *Condition Assessment and Rehabilitation of the Windy Gap Pipeline – An Owner's Perspective*, Pipelines 2001-Advances in Pipeline Engineering & Construction, Joseph Castronovo, ed., ASCE, Reston, VA (CD-ROM).

Price, Robert E., et al. (1998), *Effects of Environment on the Durability of Prestressed Concrete Cylinder Pipe*, Pipelines in the Constructed Environment, ASCE, Reston, VA, pp. 584-593.

Price, Robert E. (1990), *The Investigation, Cause and Prevention of PCCP Failures*, proceedings, AWWA Annual Conference, pp. 663-681.

Reed, Christopher, et al. (2004), *Techniques for Monitoring Structural Behaviour of Pipeline Systems*, AwwaRF, Denver, CO.

Romer, Andrew E., et al. (2004), *External Corrosion and Corrosion Control of Buried Water Mains*, AwwaRF, Denver, CO, ISBN 1-58371-347-3.

Stine, Gary P., et al. (1998), *Rehabilitation of a 183 cm PCCP with Steel Plate Liners*, Pipelines in the Constructed Environment, ASCE, Reston, VA, pp. 594-601.

Suydam, Tim, et al. (2001), *Custom Built Installation Equipment for Relining Prestressed Concrete Cylinder Pipe Minimizes Construction Schedule*, Pipelines 2001-Advances in Pipeline Engineering & Construction, Joseph Castronovo, ed., ASCE, Reston, VA (CD-ROM).

Villalobos, Jose L. (1998), *Effects of Prestressed Concrete Cylinder Pipe in a High Chloride Environment after 19 Years of Service*, Pipelines in the Constructed Environment, ASCE, Reston, VA, pp. 575-583.

Walsh, Terry L. and David S. Hodge (1998), *Overcoming the Challenges of Replacing 20 km of Defective 1524 mm Diameter PCCP*, Pipelines in the Constructed Environment, ASCE, Reston, VA, pp. 602-611.

Worthington, Will and William J. DiMarco (1996), *Acoustic Monitoring to Enhance Pipeline Safety at Crossings*, Pipeline Crossings 1996, Lawrence F. Catalano, ed. ASCE, NY, NY, pp. 1-13.

Xianjie, Kong, et al. (2005), *Condition Assessment of Small Diameter Water Transmission Mains*, Pipelines 2005, ASCE, Reston, VA, pp. 252-262.

Zarghamee, Mehdi S., et al. (1998), *Condition Assessment and Repair of Prestressed Concrete Pipe*, Pipelines in the Constructed Environment, ASCE, Reston, VA, pp. 702-713.

Zarghamee, Mehdi S (2001), *Risk Assessment and Repair Priority of PCCP with Broken Wires*, Advances in Pipeline Engineering & Construction, Joseph Castronovo, ed., ASCE, Reston, VA (CD-ROM).

Table 1 – Differences Between Lined and Embedded Type PCCP		
Feature	**LC-PCCP**	**EC-PCCP**
Diameter Range	16 through 60 inches	30 through 256 inches
Construction	Steel cylinder lined with a cast concrete core	Steel cylinder embedded in a concrete core
Prestressing Wire	Wrapped over steel cylinder	Wrapped over concrete core

Table 2 – PCCP Significant Changes with Time	
Date/Event	**Revision**
1942	First installation of LC-PCCP in U.S.
1949	First edition of AWWA C301-"Tentative", allowable wire stress approximately 45% of ultimate strength & min. mortar thickness 7/8 in.
1952	First edition of AWWA C301.
1953	First installation of EC-PCCP in U.S.
1958	Second edition of AWWA C301, allowable wire stress 70% of ultimate strength & min. mortar thickness 5/8 in.
1964	Third edition of AWWA C301, combined loading design procedure added, allowable wire stress 75% of ultimate strength.
1972	AWWA C301 revised.
1979	AWWA C301 revised.
1979	Manual M9, first edition.
1984	AWWA C301 revised, minimum coating increased to 3/4 inch, cast concrete coating deleted.
1992	AWWA C301 revised, design appendices deleted, minimum wire size increased to 0.192 inch, minimum cylinder thickness increased to 16 gauge. First edition of AWWA C304.
1995	Manual M9, second edition.
1999	AWWA C301 revised.
2006	AWWA C301 revised (draft).
2006	Manual M9, third edition (draft).

Table 3 - Reported Pipe Performance (Preliminary 1991 Data)					
Pipe Type	Length	Percent of Total Length	Failures	Failures per Mile	"Average" Years in Service
AC	165,044	1%	4	0.13	26
CI	953,183	8%	261	1.45	51
DIP	1,519,253	13%	22	0.08	15
EC-PCCP	**831,263**	**7%**	**6**	**0.04**	**22**
LC-PCCP	**2,090,605**	**18%**	**62**	**0.16**	**25**
NC-PCCP	57,560	0%	0	0	0
PT	1,534,769	13%	52	0.18	23
PVC	5,530	0%	0	0	0
RC	1,921,833	16%	59	0.16	34
RCCP	1,065,233	9%	63	0.31	25
RPM	0	0%	0	0	0
Steel	1,538,946	13%	53	0.18	34

Table 4 -Reported Failures by Size (Preliminary 1991 Data)			
Pipe Type	24 to 48 inches	48 to 72 inches	Over 72 inches
AC	4		
CI	261		
DIP	22		
EC-PCCP	**2**	**4**	
LC-PCCP	**59**	**3**	
NC-PCCP			
PT	52		
PVC			
RC	56	3	
RCCP	21	2	40
RPM			
Steel	297	6	

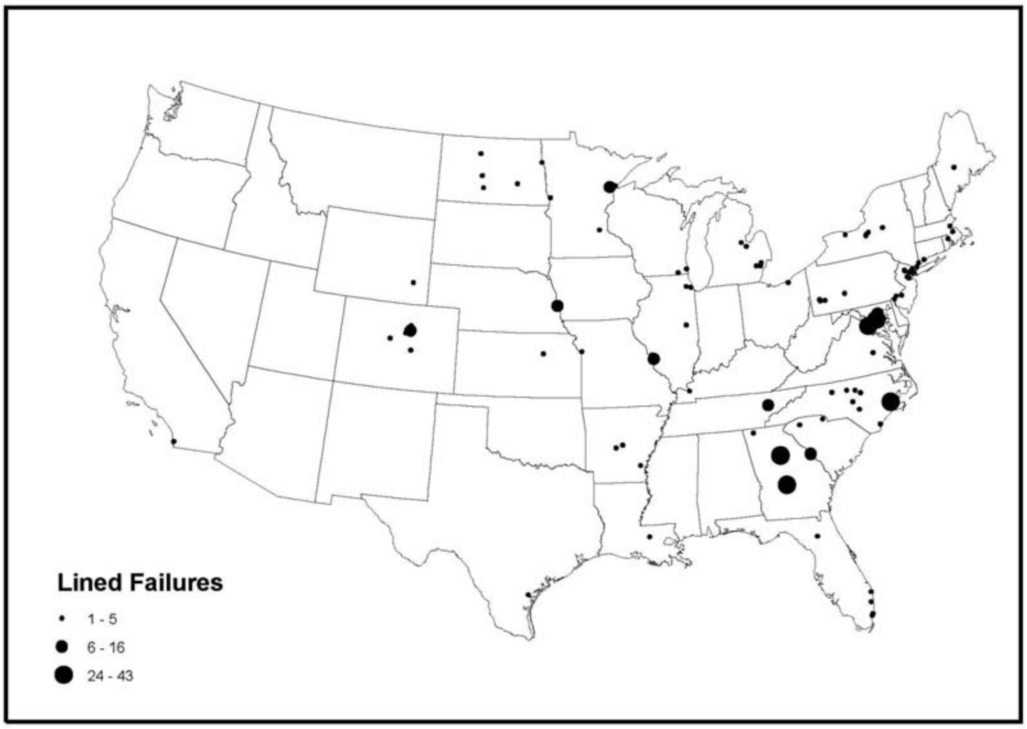

Figure 1. Location of Failures – Lined-Cylinder Type PCCP.

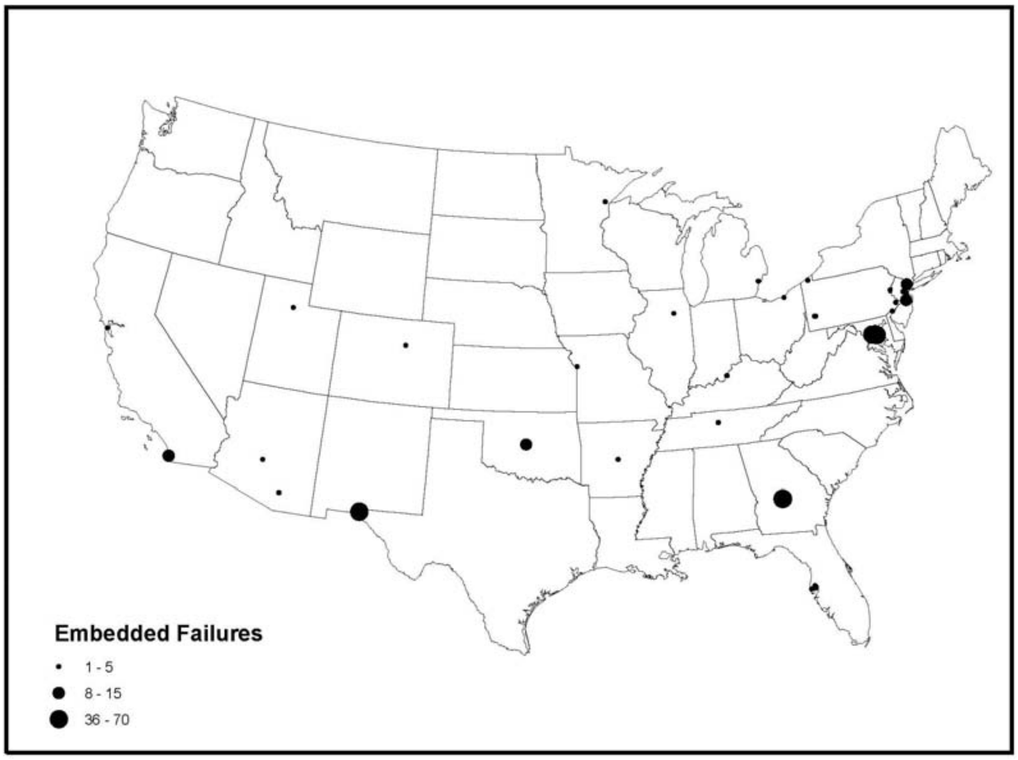

Figure 2. Location of Failures – Embedded-Cylinder Type PCCP.

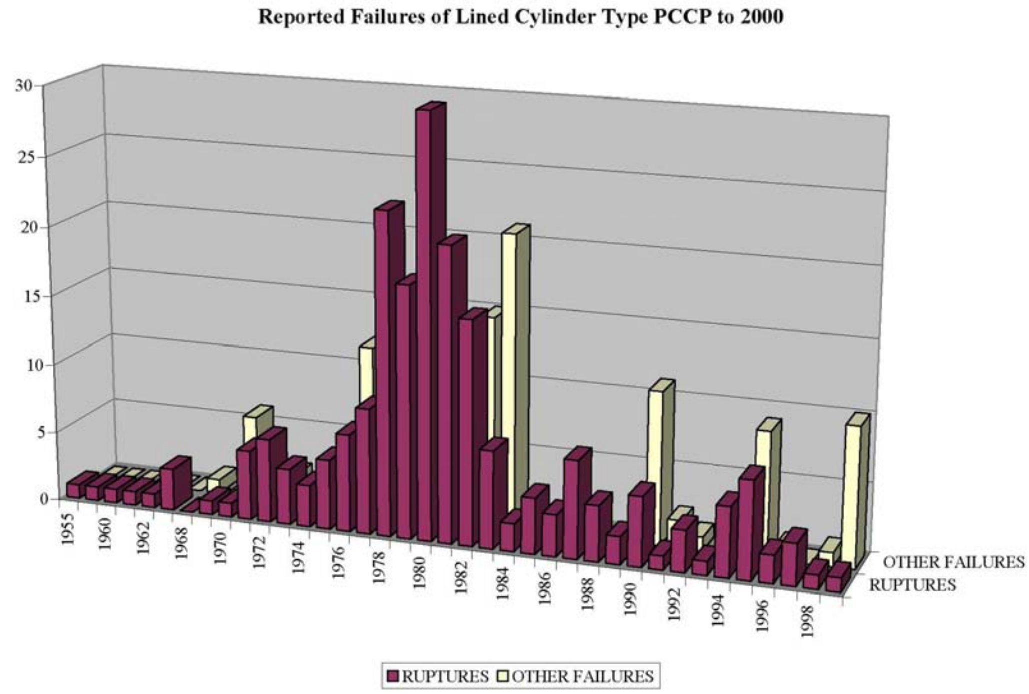

Figure 3. Lined-Cylinder Type PCCP Failures.

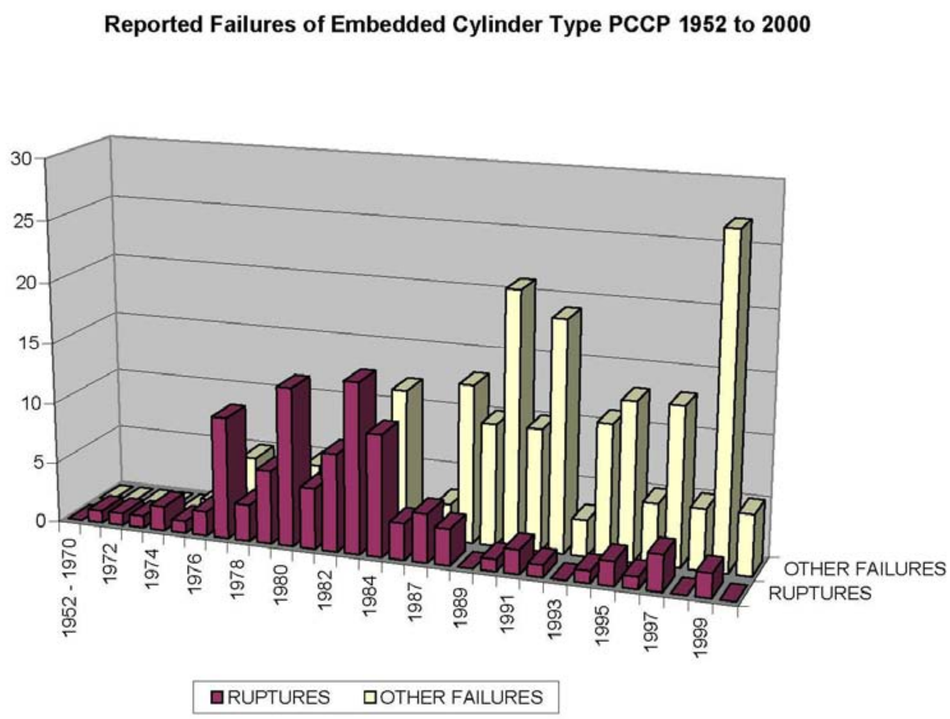

Figure 4. Embedded-Cylinder Type PCCP Failures.

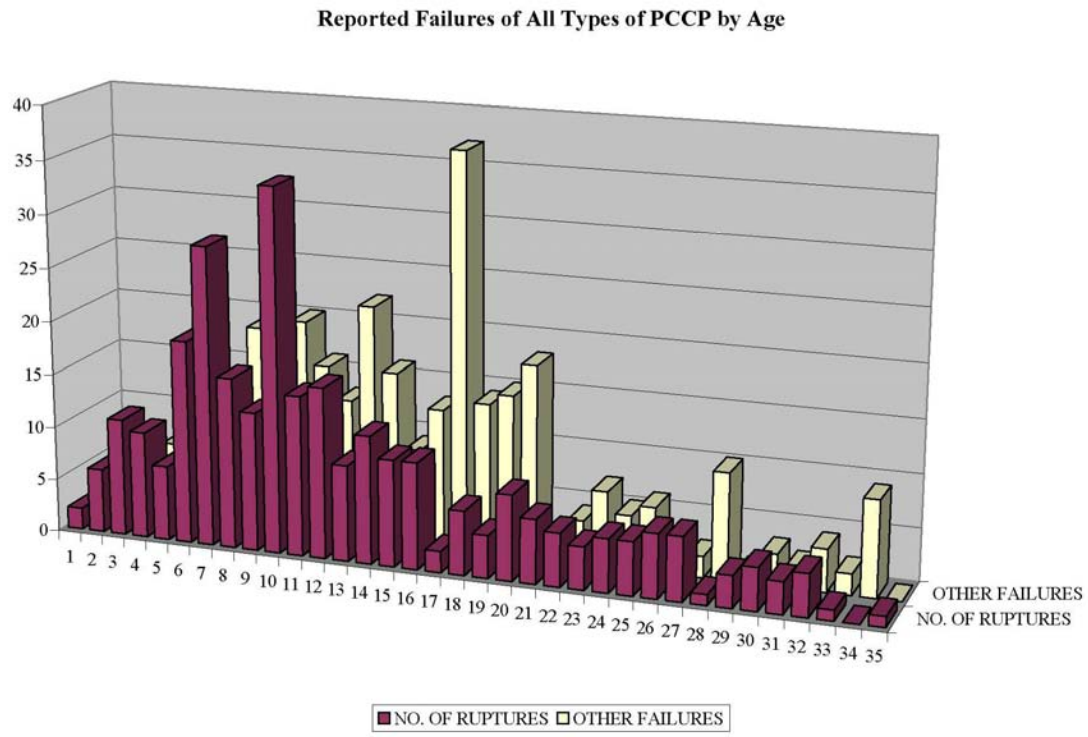

Figure 5. Summary of all Reported PCCP Failures by Age (Years) LC-PCCP and EC-PCCP.

PCCP Reliability Management

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Abstract
Pre-stressed Concrete Cylinder Pipe (PCCP) offers a reliability challenge for owners and their public. In recent years, many PCCP have been found with de-grading pre-stress wires. Pre-stress wire integrity must be maintained in order for a pipeline to remain reliable. With out fully competent pre-stress wires, life safety, property, and commerce are threatened. As a result, owners are faced with making difficult repair decisions, and providing resources required to support t on-going operation of their de-grading infrastructure.

Modern forensic technologies offer non-destruction methods to facilitate condition assessment of PCCP. These technologies allow owners to assess and categorize critical repairs with unprecedented accuracy. However, in many cases assessments have not captured all required repairs. This is due to incomplete or non-definitive assessment information, and misinterpretation of assessment data. To avoid this problem, one surefire solution has been to simply repair any suspect spools. Unfortunately, this approach requires funding that may otherwise been used for detailed investigations, more precise identification of critical repairs, or for other infrastructure improvements. Fortunately, a concise approach which utilizes modern technologies is available to assist owners with the identification of deteriorated PCCP, to make appropriate repair recommendations, and to provide appropriate monitoring for the capture of degrading performance characteristics before they become critical.

Assessment recommendations may prove to be overwhelming. Generally, the impact of a challenging repair-program not only requires allocation of unplanned funding, but may also require resources already planned for different purposes. As a result, owner's are faced with making critical operational and repair decisions, while having to manage other essential aspects of their business with fewer than required resources.

Fortunately, methods are available that streamline decision making, reduce costs, and require minimal resources, while upholding quality and safety. Assessment information for both immediate and long term pipeline-management can be joined with modern analysis methods to allow development and execution of a meaningful repair schedule. A program of this nature will relieve an owner of having to justify repairs based on vague information, and potentially sacrificing existing programs to support an overwhelming repair schedule.

The goal of any PCCP management program should be 100% reliability. Anything less would be selling the public short on water delivery and safety. Through available assessment technologies a repair schedule may be developed which holds to the ideal of 100% reliability while addressing the owner's means.

PCCP reliability is based on critical performance paths, and state-of-the-art inspection and analysis techniques. Appropriate use of these methods will allow owner's to maintain the

public's trust through a reliable water delivery system while optimizing value. Presented herein, is a method of PCCP management allowing the identification and prioritization of repairs, as well as recommendations for development of long term monitoring efforts. PCCP reliability management can be a complicated subject. Fortunately, availability of new technologies and a broad experience base has greatly simplified the process.

100% Reliability
The concept of 100% reliability has been argued as impossible and an irresponsible claim. However, comments of this nature are a result of misunderstanding. One-hundred-percent reliability articulates what an owner should be seeking, in short, perfection. Admittedly, perfection is difficult to gain. So, how can perfection exist in an imperfect world"? For the case of PCCP reliability management, perfection is achieved when a subject feeder is operating at or below its design limitation. The key word in this definition is "limitation". It's impossible to protect a water delivery system from all possible failure scenarios. On the other-hand, it is completely reasonable to expect design criteria, such as AWWA recommendations, to be met or exceeded for both new and aging pipelines. When design standards are met, a delivery system is recognized as perfect, or 100% reliable. When the design criteria are exceeded, perfection is lost. Therefore, the goal for any pipeline owner is to ensure their water deliver system operates as designed. To accomplish this, a realistic and practical PCCP reliability management program is required.

PCCP Reliability: PCCP reliability consists of three fundamental components. These are:
1) Assessment
2) Repair
3) Monitoring

The key to a successful reliability program is in understanding how to address these components. Discussions which follow provide this understanding.

Discussions
The obvious questions for any reliability program are "which pipe should be repaired and how should repairs be prioritized"? Once this hurdle is crossed, the hard work is completed. What remains is which pipe to monitor. The answer is "monitor all pipe not repaired", simple enough. However, repair recommendations can be complicated, requiring many unique assessment data to be allocated and reviewed. The Global Assessment Criteria below, **Figure 1**, lists a myriad of assessment data available to ponder. For example, compilation of assessment data such as; topographical, soil corrosivity, stray currents, water demands, and pipeline redundancy, in combination can be difficult to manage. As a result, repair decisions may become a daunting task.

1) External Inspection
    a. Visual
        i. Alignment streams
        ii. Alignment surface subsidence
        iii. Alignment greenery
        iv. Alignment proximity to private and public facilities
    b. Soil conditions

            i. Corrosivity
            ii. Stray currents
        c. Special loads
            i. Topographical
                1. earth cover
                2. slopes
            ii. Roads and highways
            iii. Railroad crossing
            iv. Buildings and bridges
        d. Wire surveys by exposures
            i. Immediate wire assessment
            ii. Extrapolated wire assessment
            iii. Magnetic confirmation
2) Internal Inspection
    a. Visual
        i. Joint cracking
        ii. Circumferential and longitudinal cracking
    b. Mechanical Sounding
        i. Manual sounding
        ii. Acoustic sounding
    c. Electromagnetic
        i. Remote Field Eddy broken wire investigations
    d. Hydrostatic and hydrodynamic
        i. Understanding design operational and surge pressures
        ii. Understanding system transients
            1. Valve closures
            2. Surge chamber requirements
        iii. Water delivery requirements and system redundancy
    e. Historical data
        i. Statistical analysis
            1. Probabilities developed

Figure 1
Global Assessment Criteria

One popular attempt to correlate the above assessment data is by assignment of index values (a numerical multiplier) for each data, setting up a mathematical relationship for the index values, then using the results of the process to make definitive recommendations. The resulting recommendations are argued as providing a relative and comparable evaluation that defines the criticality of the suspect PCCP, and also solidifies repair and monitoring decisions**, see references**.

The problem with this method is that it is based on the subjectivity of the evaluator, and not definitive assessment information. Index values are based on how strongly one “feels” the particular data relates to pipe performance. Unfortunately, feelings (and ultimately the index value) differ from person to person. As a result, consensus is difficult to gain. When consensus is obtained, the resulting recommendations are founded largely on emotions, rather than purely scientific methods and supporting definitive information.

In the end, this approach is best suited for the owner's long term planning, where a program estimate is required for the period of interest (e.g., next 10 to 15 years). When many miles of PCCP are being managed, the maintenance funding required to ensure long term operation may be several millions of dollars. Projections made with the Global Assessments will greatly facilitate a successful well planned program. Unfortunately, estimates for repair and monitoring volumes over the coming years does nothing for identification of actual repairs.

Global assessments do not have the ability to accurately identify specific repairs. Identification of detailed repairs are washed out by the myriad of assigned index-values. This results from the index-values being based on emotions and non-definitive information, and for the more elaborate programs, also includes probabilistic uncertainties. Obviously, use of non-definitive information to arrive at definitive recommendations is a contradiction. A reliability program must capture 100% of required repairs, and must be based on definitive assessments. However, it is completely reasonable to add a long term planning component to a reliability program. The long term component will be based on non-definitive assessment information, such as: Soil corrosivity, stray currents, probabilities, and other non-definitive data.

Practical Assessment

Fortunately, not all of the global-assessment data are required to make meaningful repair decisions. Identification of required repairs are easy to recognize by the grouping of information in to definitive and non-definitive categories. These categories are simply: *Repair criteria (definitive), Monitoring criteria (non-definitive).*

These criteria are subsets of the Global Assessments. This is an important concept. What is being said is "a pipe is either repaired or not repaired", nothing between. It's just that simple, you either fix it or you don't. The problem, as stated earlier, is determining which pipe to repair.

Experience tells us there are limited assessment data (definitive data) that can be realistically used to facilitate repair decisions. The Global Assessment Criteria are re-organized below, broken in to two categories (definitive, and non-definitive), and renamed as follows:

<u>PCCP Assessment</u>

1) Repair Criteria (definitive)
    a. Longitudinal wall cracking
    b. Circumferential cracking
    c. Pipe wall de-laminations
    d. Broken wire analysis
    e. Other special-case damage
2) Monitoring Criteria (non-definitive)
    a. All other assessment data

The above PCCP assessment recommendation appropriately direct repair decisions. As seen, only a few data are required for identification of repairs. All other assessment data (item 2, Monitoring Criteria) are used to help prioritize repairs for extended scopes of work, identify pipe reaches suitable for monitoring, and help predict long term repair volumes and associated costs.

The fundamental rational behind PCCP Assessment is based on the practical availability of data and its criticality to pipe performance. These data are definitive. That is, if the data exceeds a defined level, then a repair is needed. Unlike data under the Monitoring Criteria, availability of data does not define a repair, and in many cases are not practically available. For example, when pipe wall de-laminations are present a repair is mandatory, this is definitive. De-laminations either exist or they don't . De-laminations result from loss of pre-stress (failed wires) and will ultimately lead to pipe failure. On the other hand, identification of corrosive soils tells nothing of the actual pipe condition. Pipe in corrosive soils may or may not have corroded pre-stress wires. This is not definitive.

When wire corrosion is present and adverse, a significant wire cross section will be lost. When the pipe remains laminated, then pre-stress is maintained. When pre-stress is maintained, then the wire cross-section area remains (on a percentage basis) acceptable. When unacceptable, wires will break and de-laminations will present themselves. Further, it is exceeding impractical and costly to identify actual wire corrosion. To do so, one must excavate the feeder and remove the mortar coating to expose the wire. Typically, an effort of this type is used to calibrate magnetic inspections and help acquire data for probabilistic studies. This method is not used to obtain definitive repair information for extended pipeline alignments.

However, pipe in highly corrosive soils are excellent candidates for a monitoring program. In general, when corrosion is a problem, then its definitive contribution toward a repair will reveal it self through the presents of de-laminations and or the results of magnetic inspections. This fundamental concept also holds true for all other monitoring assessment criteria.

Broken Wire Analysis

Broken wire analysis is a crucial component of PCCP assessment. Data for the analysis are obtained from magnetic field surveys. The surveys allow broken wires measurements for a subject spool. The analysis consists of predicting PCCP performance, for a given pressure and cover, as it relates to the loss of pipe wall strength resulting from broken pre-stress wires. Performance predictions are the result of a combined body of information consisting of the following:

PCCP Performance with Loss of Pre-Stress

1) Wire break counts
2) Pressure
3) Cover
4) Section properties
5) Code recommendations for safe performance
6) Historical information
7) Finite element analysis
8) Failure models based on
    a. Observation
    b. PCCP geometry
    c. Testing
    d. Proven analytical methods
    e. Proprietary developments

Performance analysis with broken wires is two fold. First, the results provide a direct measure of the pipes ability to operate safely for a given condition. Second, the analysis provides a means to generate repair priorities.

Shown in **Figure 2,** are two samples of Performance Curves. Curve A is for a 54 in PCCP with 6 feet of cover and 90 psi, curve B is for the same 54" PCCP with 107 psi and 8 feet of cover. These examples demonstrate the importance of curve development for specific pipe classes and in-situ conditions. In order to competently identify a repair, a separate curve will be required for each pipe-class and loading condition. Use of one curve for multiple pipe-classes and loading conditions will likely to lead to poor repair recommendations. When the earth cover varies by more than 5 to 10% for a given pipe class, then a new curve should be developed for the subject condition. Modern Performance-Curve-Development is refined, yielding improved determinations while requiring less effort. This provides a significantly shortened response time and factional development costs compared to earlier models. The effort to develop Performance Curves is trivial compared to the value gained by their use. Curve development allows for sound repair decisions to be made. This by itself, greatly supports the reliability.

Performance plots consist of 5 curves and 4 zones. The bold colored curves (red/lower curve, blue/middle curve, and black upper curve) bound Performance Zones (1A, 1B, 2A, and 2B). The horizontal dashed line represents the operating pressure for the subject spool. The diagonal dotted line represents the effective pressure for competent wire adjacent to the subject broken wire region.

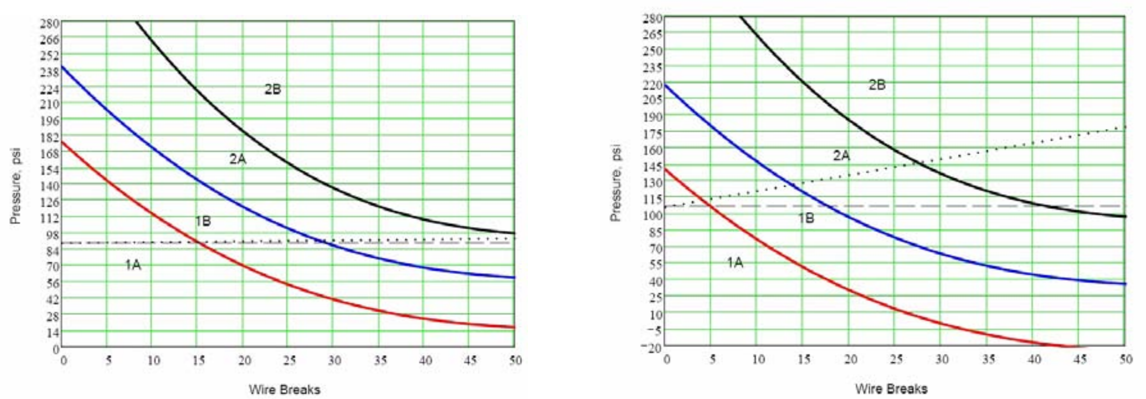


Figure 2

Performance Curves for 54" PCCP, Class 70-38: a) 6' cover @ 90 psi, b) 8' cover @ 107 psi

<u>Red/Lower Curve :</u> PCCP operating in zone 1A (below the lower curve) are with in their serviceability limit where micro-cracking is controlled. Safety factors will be below that of pipe with out broken wires. Repair of pipe in this region are considered Secondary.

<u>Blue/Mid Curve:</u> PCCP operating below the blue/mid curve, but above the lower curve are entering their elastic limit state (zone 1B) where coating cracking and pre-stress are maintained. Operation at this curve yield a safety factor of 1.0. Repair of pipe operating in zone 1B are considered Primary. Primary repairs should be performed ahead of Secondary repairs.

Black/Upper Curve: PCCP operating below the black/upper curve, but above the mid curve are approaching the ultimate cylinder and wire stress (zone 2A). Pipe performing at this curve have the cylinder and wire at ultimate stress with out the influence of earth loading. Pipe in this zone are experiencing cracked cores and coating. Many pipe continue to operate in zone 2A as a result of both wire bond and pre-stress being maintained. However, when bond and pre-stress are lost, de-laminations develop, safety factors fall below 1.0, and the pipe will fail. As more wire breaks develop the sensitivity of wire bond becomes greater (likelihood of bond-loss increases). Because of this, PCCP operating below the upper curve are considered as crucial repairs. PCCP operating above the upper curve are considered as critical repairs (wire bond is extremely-sensitive, bond is very likely to be lost in the near term).

The first point of each curve represents PCCP with out broken wires. This point should correspond to AWWA standards, and can be verified by the user. The sample curves show different initial values as a result of different earth-covers for each. As cover increases, the first curve value will be less. The converse is true for reduced earth-covers, the first point value will be greater.

All degrading PCCP should be repaired. However, the level of degradation will determine when repairs are performed. Owners should not interpret the presents of wire bond at broken pre-stress wires as an opportunity to indefinitely postpone repairs. The presents of bond at broken wires should be viewed as an interim "time-grant" best used for scheduling repairs at the earliest opportunity. The rate of de-lamination, wire breaks, and loss of pre-stress is unpredictable. *The opportunity to avoid pipe failure is significantly reduced by implementation of a delayed repair schedule.*

Identifying and Prioritizing Repairs

Pipe performance safety-factors are developed for in-situ conditions for each Performance Curve. The safety factor can be compared to an acceptable level (i.e., standard safety factor of 2.0) to gage how well a pipe is performing. With this knowledge, it's a simple task to identify pipe that are operating below an acceptable level (e.g., those having a safety factor less than 2.0).

Use of safety factors allows distinction between required repairs and those which should be monitored. That is, all pipe operating below the safe threshold should be repaired, all others should be monitored.

For many reliability programs, sole use of safety factors will yield success. However, for the special case where the scope of the repair program exceeds the means of the owner (e.g., all recommended repairs can not be done in a single shutdown), performance curves provide an additional advantage. That is, performance curves may also be used to prioritize repairs. For example, a pipe with a safety factor of 1.4 will be prioritized ahead of a pipe with a safety factor of 1.6.

A well designed pipe with out broken wires will have a minimum safety factor of 2.0. When wires are broken, the safety factor will be less than 2.0 for the same pipe. In general, pipe having a safety factor above 1.5 may be considered safe for *interim* operation. These pipes should be repaired after pipe with safety factors of 1.5 or less, but should be repaired at the earliest possible time allowed by the owner's resources.

It is highly recommended to complement prioritization with the community's interests. Accounting for the impact a pipe failure will have on the community would be a consideration of the community's interests. For example, pipe-A may have a significantly higher repair priority than pipe-B based on safety-factors. However, the decision to repair pipe-B ahead of pipe-A may be wise when pipe-B is in a public road adjacent to residences, commercial property, and an elementary school. On the other hand, Pipe-A is in rural area with no reasonable possibility of threatening life or property. Further, loss of pipe-A has no impact on water delivery, since it is readily isolated and has a parallel redundant feeder. Pipe-B, unfortunately, is the community's only water source and its loss would also impact those outside of the immediate failure zone by lack of supply.

Logistical considerations should always play an important role in repair prioritization. The specifics vary from feeder to feeder and agency to agency. Because of this, all assessment programs are well suited to having an agency's operations personnel available to facilitate repair decisions.

In summary, all pipe requiring repair will have at least one of the following characteristics: 1) longitudinal cracking, 2) circumferential cracking, 3) de-laminated zones, and 4) are operating below an acceptable threshold with or with out broken wires. Pipe not exhibiting any of these characteristics will be monitored until at least one of the above conditions are met. Repair prioritization will be complemented with logistical considerations where the interests of the service population will be best served. A flow diagram demonstrating this process is shown in **Figure 3**.

The above recommendations allow owners to focus on assessment information that are readily available and that are critical to making pipe repair decisions. Other assessment information may be used to make repair decisions, but unfortunately are impractical and costly to obtain. Further, other assessment data are not definitive. As a result these data are best suited to facilitate long term planning and monitoring programs. On the other hand, the practical recommendations for assessment are definitive, and are well suited to facilitate repair decisions.

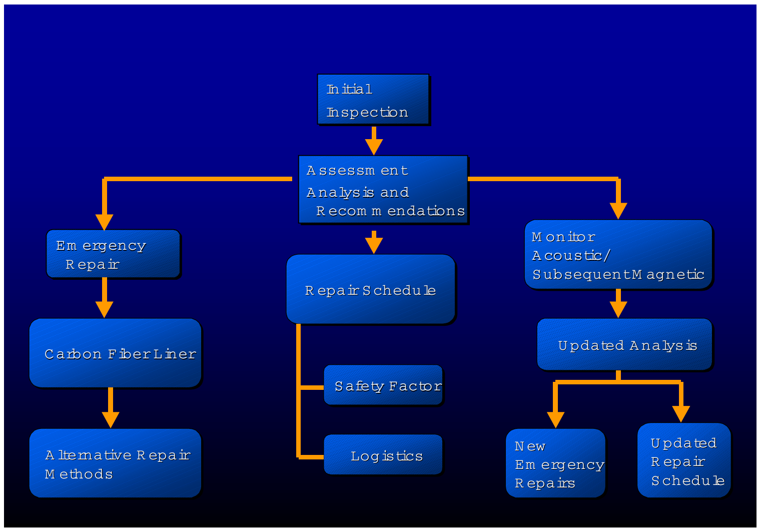
Component Discussions

Longitudinal Cracking

Longitudinal cracking is a sign the pipe has been over burdened. Cracks near the spring-line, crown, or invert are especially critical. Cracking at the crown or invert will generally be opened. These are interior tension face cracks. Cracking at the spring-line will generally be closed (hair line) resulting from compression stresses at this location. A truly overburdened pipe will contain cracks at the spring-line, crown, and invert. Opened spring-line cracks are a result of a poorly executed backfill operation during the original installation. The trench sides were over-compacted, resulting in side wall over-stress. Longitudinal cracks should be inspected for width, length, location at the circumference, iron staining, and healing by calcium carbonate. In general, pipe with longitudinal cracking should be repaired. The repair priority increases with wire breaks and de-laminations.

Circumferential Cracking

Circumferential cracking generally comes it two forms. These are spiral cracking and sectional cracking. Spiral cracking is a helical shaped crack running at the pipe circumference along the length of the interior wall surface. Spiral cracking is the result of concrete liner shrinkage and subsequent cracking at the spiral weld seams of the embedded steel cylinder. Spiral cracking initiated by the embedded cylinder's spiral weld seam are inconsequential, are hair-lined, and are usually accompanied with ontogenesis healing (calcium carbonate deposits filling the cracks).



Reliability Flow Diagram
Figure 3

However, sectional cracking is a matter for concern. Sectional cracking is associated with pipe longitudinal rotation away from the pipe joints, much like the breaking of a writing pencil at mid-body. Pipe that has rotated downward will have a larger circumferential crack at the crown compared to the invert. Pipe that has rotated upward will have a circumferential crack larger at the invert compared to the crown. Cracks of this nature are called Broken Backs and are likely to be accompanied by a failed embedded cylinder. Cylinder failure is attributed to overstress and/or corrosion. As the pipe rotates the bending stress at the embedded cylinder will cause the cylinder to rupture as its ductility limit is exceeded. Also, pipe rotation will crack the mortar coating and outer concrete core, exposing the cylinder to the environment. As a result, the cylinder will be compromised at this location by corrosion. Pipe with a broken back should be repaired at the earliest possible time.

Pipe Wall De-laminations

Pipe wall de-laminations are a direct result of local pre-stress loss. PCCP, by virtue of its pre-stressing, require the core to remain in compression. When pre-stress is lost, the core expands and looses intimate contact with the cylinder (i.e., de-lamination develops). Pre-stress loss weakens the pipe at the de-laminated zone. Stresses are redistributed from the de-laminated zone to adjacent competent wires. With time, the competent wires will loose pre-stress due to breaks, mortar creep, and over stress caused stress redistribution. De-laminations will propagate, and ultimately fail the pipe. De-laminations are a red flag signaling loss of wire bond and pre-stress. De-laminated pipe should be repaired at the earliest possible time.

Broken Wire Analysis

The heart of Performance Curve development is proprietary. However, the methodology also contains components that are non-proprietary. Such as the implementation of guidelines set by AWWA, use of the Finite Element Method, and publicly available pipe failure information. In general, performance predictions of PCCP with broken wires consists of proprietary technology, the finite element method, field observations, testing, wire break counts, original pipe cross-section information, and field conditions (earth-cover and pressure).

The fundamental approach is to first identify the wire break count by use of available magnetic technologies, then to run the analysis based on the above information. An operational safety factor for the specific wire break count will be developed, provided the wire break information is made available. When wire break information is not available, safety factors will be made for wire break counts at transition zones defined by the operating pressure. In general, PCCP with out broken wires will have a safety factor of 2.0. This information is useful for comparisons to operating conditions at zero wire breaks. As communities develop, grade changes over the pipe may occur, and water delivery requirements may differ than originally intended. These varied conditions may either increase or decrease the existing safety factors compared to the original installations. When pipe are operating with safety factors below 2.0 for in-tact wire, then more caution should be taken as broken wires are discovered. On the other-hand, when in-tact pipe are operating with safety factors above 2.0, then less caution is required as broken wires are uncovered. Caution is always recommended for pipe containing broken wires. The level of caution (i.e., repair now or repair later) will depend on the recommendations of the engineer in charge and the pipe owner.

Summary

The myriad of assessment information available for the creation of a reliability program can be overwhelming. Fortunately, definitive assessments are available to create a competent reliability program. These data provide a concise and cost effective method for managing an aging PCCP infrastructure for both near and long term purposes. Definitive assessments facilitate the need for repair and the development of repair schedules. Use of non-definitive assessments, provide information needed to facilitate long term planning and resource allocation. These are vital components of a comprehensive reliability program.

References

Muhlbauer, W. Kent, Pipeline Risk Management Manual (2nd ed.), Houston Texas: Gulf Publishing Company, 1999.

Ameron (1986). “Prestressed Concrete Cylinder Pipe.”, Design Manual 301, Monterey Park, California.

AWWA (1999). “Design of Prestressed Concrete Cylinder Pipe.”, AWWA C304-99, Denver, Colorado.

Welded Lap Joint Brittle Failure: A Structural Assessment of an Atlanta 72-inch Welded Steel Water Pipe Demonstrates Need for Improvement in AWWA Standards

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Abstract

A condition assessment of the City of Atlanta 72-inch buried welded steel raw-water pipeline was performed in 2006 by CH2M HILL/Williams Russell & Johnson Joint Venture Inc. and ATS Inc. on behalf of the City of Atlanta Department of Watershed Management. The pipeline is spiral welded with ½-inch wall thickness and double welded bell and spigot lap joints. It was fabricated in 1973 and 1974 to the standard of "Mill-Type Steel Water Pipe," AWWA C202-64T, Class B. This assessment focused on structural integrity, as the pipeline has a history of numerous failures since entering service in 1975. Past failures have been reported as full-circumferential brittle fractures through the bell at the toe of the internal fillet weld. In-situ and laboratory structural examinations were performed including extensive analyses on a 4.5-foot sample, including an intact bell and spigot lap joint, cut from the 72-inch welded steel pipe (WSP) alignment. This assessment did not include an investigation of an actual failure site.

Data from this assessment indicates that the steel is extremely brittle within the cold-formed bell (Charpy V-Notch < 6 ft-lbs at 60º F). Past failures are likely the result of a high concentration of stress in the bell at the toe of the internal lap (fillet) weld of the fully stabbed joint, where the pipe steel is extremely brittle and where welding-induced notches were observed. All failures have occurred during winter months when water temperature is lowest, the steel is most brittle, and thermal induced longitudinal tension is maximized.

The authors wish to translate the results of this condition assessment into recommendations—from the consumer's perspective—for the improvement of the industry standards for welded steel water pipe. Although AWWA has made improvements to their standards (M11-2005, C200-1997, and C206-2003) due to lessons learned from other WSP failures, gaps remain within these publications that may allow defects in design, fabrication, and installation of a new WSP.

72-Inch WSP Design and Construction History

The 72-inch WSP alignment operates in parallel to a triplet of cast iron pipelines installed between 1893 and 1924. All pipelines convey raw water pumped from the Chattahoochee River to the Hemphill Reservoirs in Northwest Atlanta. In 1994 the steam pump station servicing these pipelines was converted to an all-electric station and an additional surge tank was added. Installation of the first segments of the pipeline commenced in June of 1973 and the pipeline entered service in 1975.

The 1972 design engineer's specification (Wiedeman and Singleton, 1972) for the 72-inch WSP required ½-inch wall thickness and steel conforming to AWWA C201-66, ASTM A-283 Grade D, or AWWA C202-64 Grade B as fabrication standards. The field assembled joints were stipulated to be double welded bell and spigot type or butt straps at closure joints—butt welded joints were prohibited.

The original design specifications call out that joint restraint and longitudinal stress calculations shall be based on an internal pressure of 175 psig, a post-installation hydrostatic test pressure of 160 psig, and a shop hydrostatic testing standard for each pipe segment of 250 psig. The pipe was manufactured by Lone Star Steel of Lone Star, Texas to the standard of AWWA C202-64 Grade B, which has minimum yield strength of 35,000 psi.

The design specification does not address the issue of pipeline installation temperature or the expected service temperature. The installation and field welding standard cited was C206-62, which does recommend general measures to minimize installation temperature and thermal stress but provides no requirement. A 2005 conversation with a City of Atlanta engineer, who observed the 72-inch WSP installation, revealed that there may not have been consideration of pipe temperature upon welding or back-filling the lap joints.

The pipeline has a factory-applied coal-tar enamel and felt wrapped external coating system, and a field-applied ½" thick cement mortar interior lining. The bell ends were reported to be cold formed with an expansion device and not rolled with an offset belling die.

Failure History

Seven failure events are reported for the Atlanta 72-inch WSP (1980 x 2, 1982, 1993, 1994, 1999, and 2003); however, corroborating evidence does not exist for the location or mode of failure for three of these events. The character of the four documented failures is similar: full circumferential fractures through the bell at the toe of the internal fillet weld.

All known failures have occurred in winter months, December, January and February (the exact date of some events is not known), and cold water temperatures are well correlated with the incidence of pipeline failure. Extrapolations from a USGS Chattahoochee River monitoring site downstream of the intake suggest median winter (December, January and February; 1976 to 2006) minimum daily water temperature is approximately 48° F. All failure events are believed to have occurred when minimum daily water temperature was 42° F or less.

The City of Atlanta operations and engineering staff have long suspected that water hammer was a factor in the failure of the 72-inch WSP. The 72-inch raw water WSP is fed by a pump station located at the bank of the Chattahoochee River. The pipeline terminates at the Hemphill Reservoirs, approximately 5 miles from the pump station and approximately 230 ft. higher elevation (100 psig static). The coincidence of a complete power failure (the maximum surge condition) and a pipe failure has not been reported. Failures of the 72-inch WSP are believed by City of Atlanta staff to be related to a single pump shut down or valve closure event. The magnitude of a single pressure transient event resulting from a 40 MGD pump shutdown was measured in 2006 with a portable transducer and data logger. Pressure data for event were recorded once per second as an average of four ¼-sencond readings. The transient was measured approximately 2100 feet downstream of the pump station and generated a 20 psi increase in pressure above steady state and 40 psi below steady state—well within an acceptable range for a typical WSP.

Prior Failure Analysis

A laboratory investigation into the cause of a 1982 failure of the 72-inch steel water main, the third failure, was conducted and remained the primary reference for the condition of the pipeline steel for more than two decades (MEA, 1982). The report concluded that a full-circumferential brittle catastrophic failure was "the combined result of poor notch toughness (impact) properties of the steel, high stresses in the joint area, possibly an initiating notch at the spiral weld and girth weld intersection, and a sudden impact loading (probably due to water hammer)." The report also cites a review of a previous failure analysis and information from a City of Atlanta engineer to conclude that two 1980 failures were "virtually identical" in nature to the 1982 failure. The report was not considered fully conclusive due to uncertainty with the origin and orientation of the Charpy impact test specimens and omissions identified in the joint stress analysis. The conclusion of a sudden impact load appears to be speculative.

Rationale for Study

The primary objective of the intensive 2006 investigation was to compare and contrast the metallurgical properties of the cold formed bell region vs. the base metal of the pipe barrel with an eye towards potential structural rehabilitation. If it could be established that the structural defects were limited to the lap joint, exclusive of the base metal in the pipe barrel, it may be possible to bypass the load-path around the defective bell joints. Such a design would include an internally welded butt-strap and has been successfully applied elsewhere to rehabilitate WSPs where joint strength was inadequate.

Condition Assessment Scope Summary

The Atlanta 72-inch WSP condition assessment investigation included: a corrosion assessment, evaluation of the cathodic protection system, examination of coating and liner systems, soil corrosivity evaluation, a basic geotechnical study, internal remote camera inspection (12% of alignment), in-situ non-destructive external and internal structural examination, and laboratory structural evaluation including metallurgical and mechanical testing. The scope of this paper is focused on the in-situ and laboratory structural evaluation—which was the primary focus of the 72-inch WSP condition assessment. A companion paper to this document, published in these proceedings (Hunt et al., 2007), provides an overview of the City of Atlanta Raw Water Transmission Pipelines Condition Assessment.

In-Situ Non-Destructive Examination Scope

For the in-situ structural assessment, external non-destructive examinations (NDE) were performed at four sites along the alignment and internal NDEs were performed at three of these sites. The excavation sites for external inspection were restricted to unpaved areas close to the internal access points. Although not equally distributed along the alignment, the excavations sites were separated at least ½-mile. One site was intentionally located in the most corrosive soil detected. Approximately 50% of the 5-mile alignment was dewatered to allow the internal assessment, radiographic examination of welds, and removal of a 4.5-foot long sample section.

The internal condition assessment task was performed at six bell and spigot lap joints, two joints within a short distance of each of the three access points. At each joint, the cement lining was removed to inspect approximately 2 square feet of pipe surface at three locations around the pipe inner circumference (Figure 1). Additional liner was removed if a feature of interest was uncovered, such as the factory spiral weld or a factory stencil marking.

FIGURE 1
Typical Internal Surface Preparation for Non-Destructive Testing

The target surfaces for the internal NDE (figure 2) were the area immediately surrounding the field-assembled bell and spigot lap joint and small sections of the pipe barrel that are traversed by a factory spiral weld. Six to eight sections of external coating were removed at each site, with each removed section varying in size from 0.5 square foot to 3 square feet. Approximately half of these sections exposed the bell and spigot joint.

FIGURE 2
Typical External Surface Preparation for Non-Destructive Testing

Visual examination (VT) was performed on all internal and external exposed surfaces to measure weld size, identify any welding-related defects that could serve as stress risers, and identify any evidence of excessive cold working/cracking. VT was enhanced by magnetic particle testing (MT) and penetrant testing (PT) over all exposed surfaces. Pipeline wall thickness was measured with ultrasonic thickness testing (UT) to determine if the pipeline had experienced wall loss from corrosion or erosion during its service life. Measurements of bell geometry were collected and observations were made of bell and spigot fit-up, such as depth of spigot insertion and eccentricity of the spigot within the bell. Bell geometry was measured with a profile gauge, a device commonly used in the wood-working trade that allows vertical and horizontal measurement of bell curvature. On-site radiographic testing (RT) of was performed in conjunction with the external assessment.

Laboratory Destructive Metallurgical and Mechanical Evaluation Scope

To allow a structural investigation of a complete bell and spigot lap joint and of the pipeline barrel on both sides of the joint, a 4.5-foot sample section of the 72-inch steel pipeline was saw-cut and removed from Station 21+00 (Figure 3). The removed section was replaced by Rockdale Pipeline Inc. with made-to-order spool manufactured on 4-day notice by Northwest Pipe.

<u>Visual Observations and Measurements</u>

- 18-inch long strips were cut through the bell and spigot joint, along the pipeline axis, at 3 locations around the circumference. See Figure 4 below for locations of all test specimens removed from the 72-inch steel pipeline sample section. The bell and spigot sections have been separated for clarity of presentation. Figure 4 is not to scale.
- Macro-etch examination was performed on portions of each axial section/strip to allow detailed observations of base-metal condition, weld quality, weld position, and bell and spigot fit-up conditions.
- Detailed measurements were taken of each sectional strip with a coordinate measuring machine.
- Observations were made regarding field welding processes and techniques.

FIGURE 3
4.5-ft Sample (with complete lap joint) cut from 72-inch Pipeline for Lab Analysis

<u>Mechanical Testing</u>

- Micro-hardness testing was performed on traverses across multiple factory- and field-welded connections to reveal the condition of the heat-affected zone (HAZ) relative to the weld metal and the pipe barrel metal.
- Numerous Charpy impact tests from four sample locations were conducted at a range of temperatures on full-size (10-mm x 10-mm) specimens. The sample locations included two locations on the pipe barrel: one each from the spigot piece and the bell piece that were not subjected to the cold working by bell-end expansion. The two other sample locations were from the cold formed bell region: one from the toe of the internal fillet weld on the curved portion (partially including the HAZ) and another from the flat portion near the end of the expanded bell (zone of maximum diameter).
- Tensile strength, yield, and elongation tests were performed on the samples cut from the pipe barrel on both the bell and spigot pieces in varying orientations.

<u>Metallographic Evaluation</u>: Micro-structural observations were made of samples from the cold worked bell region, the pipe barrel, and the HAZ of the factory spiral.

Chemical Analyses were performed on samples (2) from the bell piece and the spigot piece.

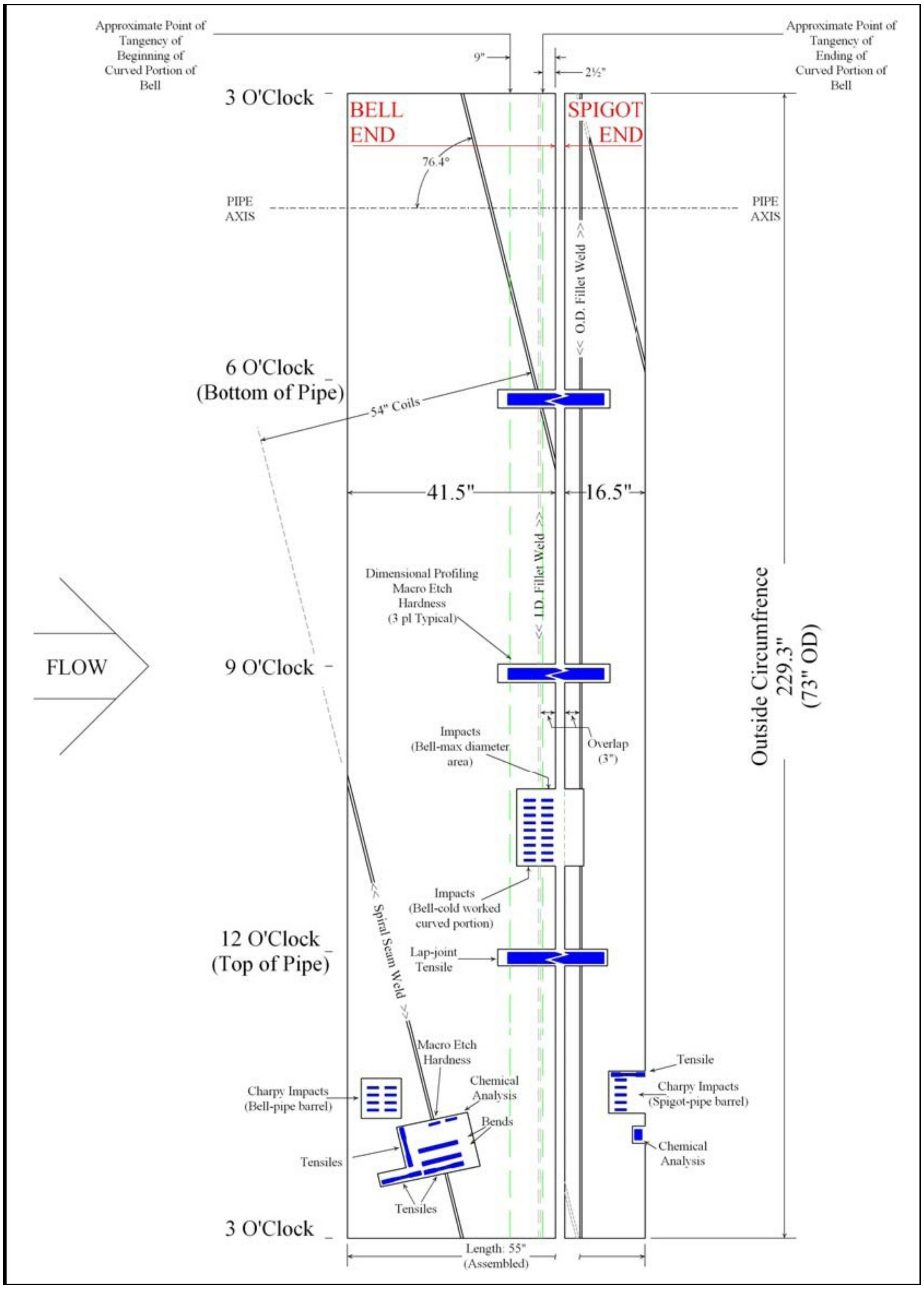

FIGURE 4
Developed and Disassembled View of 4.5' Pipe Spool Showing Specimen Locations

Summary of Assessment Results

Measurements of the external profile of the pipeline bell confirm that the bells were likely formed with a segmental expander resulting in a radius of curvature exceeding 7.5 inches, or fifteen times the wall thickness (t) of the pipe as recommended by the current AWWA C200-97 standards for welded steel pipe—no standard for bell radius of curvature existed in 1973. It is widely assumed that a minimum radius of curvature of 15*t will prevent significant degradation of notch toughness during bell formation.

External and internal inspection of the lap joint fit-up revealed that the pipe joints were fully stabbed, or very nearly fully stabbed. Bell and spigot overlap was found to be 3" to 5” on the inspected joints. The design drawing for bell assembly in the original construction documents shows a fully stabbed joint. This installation technique positioned the internal fillet weld directly on the curved bell transition. Full insertion of the spigot into the bell, and welding on the bell transition, is now prohibited by AWWA C206-03.

Measurements of the gap between the outer diameter (OD) of the spigot and the inner diameter (ID) of the bell for the sample taken from station 21+00 show that an effort was not made, during installation, to equalize the gap around the pipe circumference for this straight pipe section. A gap exceeding 1/8-inch between the bell and the spigot exists at the 12 O’clock position while no gap exists on one of the sides and bottom of the pipe (Figure 5). This off-center fit-up and gap amplifies an eccentric load-path that concentrates a substantial stress at the root of the fillet welds. Present day AWWA standards limit this gap to 1/8-inch; however there is no mention of equalizing the gap around the circumference.

VT, MT, and PT revealed no evidence of cracking of the pipeline steel at the bell and no evidence of cracked welds.

Macro-etch examination of the field fillet welds revealed that they were welded using a shielded metal arc welding process—electrode type unknown. The multi-pass, internal fillet welds began at the top of the pipe whereas the single-pass, external fillet welds started at the bottom. The external and internal fillet welds at the lap joints (Figure 5) were found to meet the 5/16-inch and 3/16-inch size requirement of the original specification, but some of the fillet welds did not have an acceptable profile due to weld undercut or overlap. Although incidences of undercut and overlap are notable due to their potential to serve as stress concentration and crack initiation sites, the majority of the incidences observed did not exceed welding code tolerances in effect at the time of construction (AWWA C206-62) or welding code AWS D1.1-2006 for statically loaded non-tubular connections.

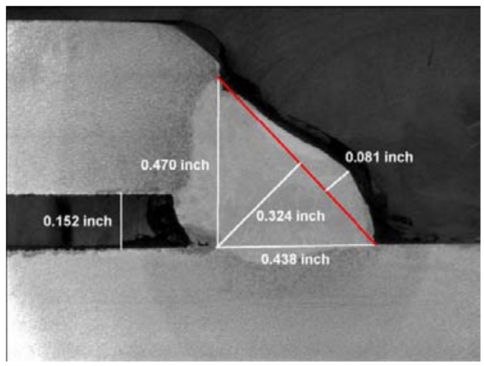

FIGURE 5
Polished and Etched X-Section of the External Fillet Weld at 12 O’Clock Position at Station 21+00

Numerous arc strikes were found throughout the bell and spigot region of the pipeline. An arc strike is a welding-related blemish caused by the activation of the welding arc on the surface of the pipe barrel away from the weld area. The locations of un-repaired arc strikes could serve as crack initiation sites for brittle pipeline failures. A contemporary welding code stipulates that all such blemishes should be ground smooth and inspected for cracks (AWS D1.1-2006, Paragraph 5.29)—however this portion of AWS D1.1-2006 is not referenced by AWWA C206-03.

VT, PT, MT of the factory spiral welds indicated that they had acceptable profiles and reinforcement as per meeting current welding code (AWS D1.1-2006, Section 5.24.4). However, RT revealed that portions of these welds were unacceptable due to their porosity, incomplete fusion and penetration, and inclusion of slag. Although these defects are notable, they are not likely related to the incidences of full-circumferential brittle failure.

UT wall thickness measurements indicate there has been no significant erosion or corrosion of the steel material over its service life.

Micro-structural observations of both the pipe barrel and cold-worked regions of the steel bell revealed a banded ferritic-pearlitic microstructure--typical for hot-rolled steel. The microstructural exam did not show any significant difference between the body and cold-expanded section of the bell.

Tensile testing of the pipe barrel found it to meet requirement of AWWA C202-64T Grade B, and to be similar to currently acceptable classes of structural steel for steel water pipe, such as ASTM A1018, Grade 36, Type 1. The cold formed portion of the bell was not evaluated. Results from tensile tests are provided in Table 1. Tensile testing and bending of the factory spiral weld found it to be acceptable and, in fact, stronger than the pipe barrel.

Chemical analysis of samples revealed the pipe met the chemical requirements for AWWA C202-64T Grade B steel and is similar (carbon content was slightly higher) to the specification for hot rolled structural steel ASTM A1018 Grade 36 Type 1.

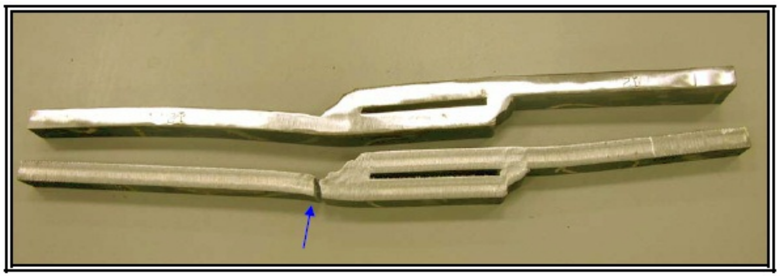

FIGURE 6
Position of Lap Joint Brittle Failure at Toe of Internal Fillet Weld

Additional tensile testing of a ¾" wide section of pipe cut through the lap joint at the 12 O'clock position was found to fail in a brittle manner at the toe of the internal fillet weld at a stress of approximately 30% less than the tensile strength of the pipe material (Figure 6). No fillet weld defects were present on this ¾" wide section. Observations of the fracture surface show that a majority (~90%) of the fracture surface was reflective—an indication of brittle fracture (see Figure 7). There was a small ductile-appearing zone that was most likely the weld HAZ; this suggests that the heat input from the welding process improved the properties of the brittle material in this area. While this joint tensile test was not meant to simulate the failure of the pipe under normal loading conditions, with resistance to bending provided by the full pipe ring, it does indicate this location to be a weak link.

Hardness testing through the pipe barrel, heat affected zone, and welds yielded hardness values typical of low carbon steel, consistent with the chemical composition described above. The measured hardness of the all samples was found to be well within acceptable limits; the heat affected zone at ID fillet weld was not excessively hardened.

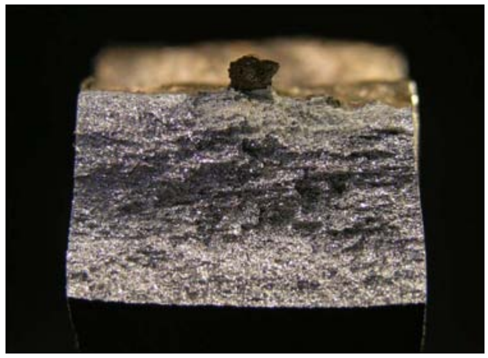

FIGURE 7
Close-up view of the lap joint fracture surface with a small ductile zone in the upper-right.

Charpy V-Notch testing of the two pipe barrel samples taken from the 4.5-foot spool had a notch toughness of 15 ft-lbs at 60ºF and < 10 ft-lbs at 40ºF. CVN tests performed on the cold worked region of the bell show a notch toughness of 15 ft-lbs at 110ºF and less than 5 ft-lbs at temperatures under 60ºF. Weld metal toughness was not evaluated. Energy transition curves for the four sample locations are plotted in Figure 8. Data from a 1982 analysis of the same pipeline (MEA, 1982), including numerous CVN specimens from three samples of steel from two prior failure sites, show all CVN values < 4 ft-lbs at 60 ºF. The finding of low notch toughness was common to other notable failures of steel pipelines with field-welded lap joints: the 82-in. ID (15/16-in. wall thickness) Second Los Angeles Aqueduct failures in 1970 (Phillips, Triay, and Marynick, 1972) and the Denver Water Board 108-inch (½-inch wall thickness) pipeline failures in the early 1980's (Eberhardt, 1990); however, the notch toughness results for the Atlanta 72-inch pipeline are the lowest of the three.

TABLE 1
Tensile Properties of 72" Steel Pipeline

	Results Bell			Results Spigot	Steel Pipe Production Standards	
	Longitud-inal	Trans-verse	Spiral Weld	Parallel to Pipe Axis	AWWA C202-64 Grade B	ASTM A1018-05 SS Grade 36-1
Tensile Strength, psi	70,000	69,500	75,500	69,500	> 60,000	> 53,000
Yield Strength, psi	39,200	40,100	50,500	40,200	> 35,000	> 36,000
Elongation, %	32	28	19	31	N/A	> 21

Discussion of Steel Notch Toughness

The AWWA fabrication standard identified in the original design specifications did not include a requirement for notch toughness. In 1975, the AWWA adopted a revised standard (C200-75) for steel water pipelines, which included a general warning but not a specific standard regarding the significance of notch toughness in the design of steel pipelines with wall thickness greater than ½".

The importance of notch toughness was known in the steel pipe industry prior to 1973, the year fabrication of the Atlanta 72-inch WSP began; however, it appears there was an assumption that the property of notch toughness did not require special consideration for steel plate or coil ½-inch thick or less. A well known July, 1972 publication "Pipeline Problems—

Brittle Fracture, Joint Stresses and Welding" (Phillips, Triay, and Marynick, 1972), declares the importance of steel toughness in water pipeline design and warns about weld quality and stress concentration at the lap joint. The paper discusses the lessons learned from the Los Angeles Department of Water and Power's (LADWP) 15/16-inch thick 82-inch diameter WSP and the authors quote a thickness threshold of greater than ½-inch for focused attention on the property of notch toughness. One is left to assume that manufacturers inherently produce notch tough steel in the thickness range of ½-inch and less or that the potential for brittle fracture is less with thinner wall pipe. Unfortunately the failure history of Atlanta's 72-inch WSP and Denver's 108-inch WSP suggest otherwise—at least for steel produced circa 1973 and 1980.

The assumption that steel ½-inch thick and less does not require specific quality control for notch toughness is still common in the water industry today. Both AWWA C200-97 *Standard*

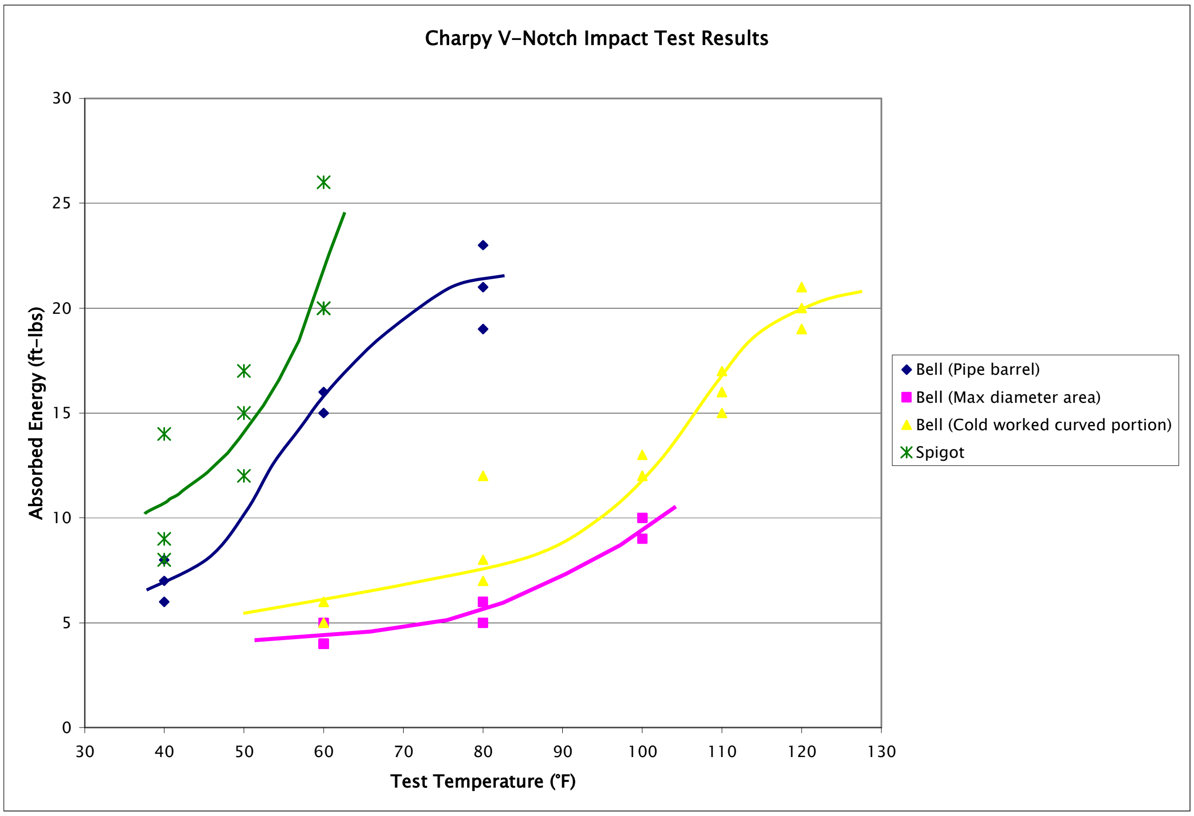

FIGURE 8
Charpy V-Notch (CVN) Impact Test Results

for Steel Water Pipe—6-inch and Larger, and AWWA M-11-05 *Steel Pipe—A Guide for Design and Installation* do not provide specific guidance for the specification of adequate steel notch toughness for wall thickness of ½-inch and less. Even though steel fabrication quality has improved since 1980, and many high quality steel manufacturers routinely produce steel with adequate notch toughness for cold water service, the pipeline purchaser is still at some risk of receiving a low-notch tough product when an explicit requirement does not exist. Considering that steel with a CVN value exceeding 25 ft-lbs at 32° F is commonly available from today's steel industry, the cost of such a standard for the steel pipeline manufacturer or purchaser is not significant relative to the benefit. The master specifications for several large U.S. engineering firms already close this gap in AWWA standards.

The issue of adequate steel toughness is not only related to the topic of pipeline base metal specifications; steel handling and pipe manufacturing techniques also control the final toughness of the pipeline steel. The CVN results from this study put in question the assumption that a radius of curvature for bell-end expansion of 15 times wall thickness (t) will not significantly degrade steel toughness during bell formation. The 15*t standard in AWWA C200-97 includes no stipulation on how bell formation shall occur, by rolling or otherwise, and formation of the bell is not the only cold working process involved in pipeline fabrication. A steel pipe fabrication standard that provides a basic minimum notch toughness standard for the finish product, and a basic enforcement scheme, would vastly improve the certainty that all structural members within the load path perform as assumed in the design calculations. Such a post-fabrication notch toughness standard should supplement the existing 15*t minimum radius of curvature.

Discussion of Lap Joint Design

The results of this study suggest that the bell can be a point of weakness for an internally welded bell and spigot lap joint. The observed weakness of the Atlanta 72-inch WSP lap joint is likely the result of low notch toughness through the cold worked bell and the fact that stresses are concentrated in this brittle region of the pipeline due to the eccentric cross section of the lap joint—see figures 5 and 6 for images of the cross section. Stress concentration through this kind of joint is discussed in detail by Phillips, Triay, and Marynick (1972), Eberhardt (1990), and Brockenbrough (1990).

Several factors affect the magnitude of stress concentration in the bell of a lap joint including: radius of curvature of the bell transition, offset of the internal fillet weld from the curved portion of the bell, the size of any gap between the bell and spigot faying surfaces, and an unequal gap between the bell and spigot around the circumference. Any axial deflection of the spigot inside the bell, to achieve a long radius curve, effectively creates a gap between the bell and spigot. The presence of defects in the fillet weld profile such as undercut, overlap, or excess convexity can further concentrate stress at a given point on the circumference and serve as a crack initiation site. Current versions of AWWA documents referenced by this paper have improved, since 1973, their regulation and discussion of several of these practices; however their individual or cumulative impact on joint strength remains unquantified.

Similar defects were present in the lap joint design of the Atlanta 72-inch WSP and the two other welded steel pipelines referenced in this paper: the Second Los Angeles Aqueduct 82-inch and Denver Water Board 108-inch. In each case the lap joints combined brittle steel with excessive stress near the internal fillet weld. It appears that brittleness of the steel through the bell precluded local yielding and stress relief that could otherwise prevent brittle fracture at a joint with adequate notch toughness and ductility. Such a combination of defects at the joint suggests that the decision to use bell and spigot lap joints in these pipelines was the result of adherence to industry-standard practice rather than a quantitative design effort. Eberhardt (1990) makes this point in his reference to the Denver Water Board 108-inch by ½-inch WSP failure history, the "…bells are a product of many years of usage and of fabricating process, but not a product of a design office."

In addition to Eberhardt, other researchers have concluded that the bell of the welded lap joint is not as strong as the pipeline barrel, (Brockenbrough, 1990) and (Phillips et al., 1972). Data published by Brockenbrough, for an internally welded bell and spigot lap joint, suggest the bell is less than half as strong as the pipe barrel (<50% efficient) due to the stress concentration through the eccentric load path—other published results vary. Regardless of the precise value, current AWWA M11 or C200 documents do not provide any guidance related to the ultimate strength of the bell and spigot lap joint or any other shop or field welded joints commonly employed in steel pipelines.

The failure histories of the three welded steel pipelines discussed in the paper exemplify the potential consequences of omitting quantitative principles in joint design. The pipeline designer must be able to quantify the strength of a chosen joint configuration in order to properly calculate maximum axial design stress for that member with a known safety factor. To facilitate this calculation, a fully documented design approach for the bell and spigot welded lap joint should be included in AWWA M11. With a standardized design approach, the bell and spigot lap joint will likely continue to be the most commonly specified field joint for steel water pipe—with the added benefit of lower risk of failure for the future pipeline owner.

Discussion of Pipeline Axial Stress

Because the pipeline was likely installed without adequate temperature control through all seasons, it is possible to assume that at least portions of the alignment are under significant thermal-induced tension when the water temperature is low—even though it is not possible to resolve the exact magnitude of these forces. Considering that the Atlanta 72-inch WSP failures have only occurred when water temperature is approximately 42° F or below, while the steel of the bells remains brittle at temperatures well above 100° F, it is logical that thermal axial stress must be a factor in the joint failures observed. Routine water hammer alone does not appear to be sufficient to cause brittle fracture of this pipeline.

It is notable that a thorough discussion and concise listing of formulas for resolving total axial stress on a WSP are missing in the current version of AWWA M-11-05 "*Steel Pipe—A Guide for Design and Installation*," (stresses due to temperature change, Poisson's effect, soil drag, or PA force). This is a basic step in steel pipeline design, a passage in AWWA C206-03 "*Field Welding of Steel Water Pipe"* even states it is up to the designer to calculate and manage potential thermal stresses in the design of a WSP. Improving guidance in AWWA M11 for the calculation and control of axial stress in a WSP would be a positive step to prevent a loading mechanism that contributed to the failure of the Atlanta 72-inch WSP.

In addition, the development of a standard method for determining local pipeline temperature during installation would improve enforcement of specified thermal controls and increase the likelihood that an installed pipeline experiences a range of axial stresses similar to design assumptions.

Conclusions and Recommendations

The repeated failures of Atlanta's 72-inch WSP bell and spigot lap joints appear to have been triggered by a combination of shortcomings in design, manufacturing, and installation. High axial thermal tension, or possibly high thermal tension in combination with a water hammer event, is likely the loading mechanism that ultimately triggered the past failures. The eccentric geometry of the lap joint acts as a stress riser immediately at the toe of the internal fillet weld; this stress riser effect is amplified by the location of the weld on the curve of the bell transition, the presence of any gap between the faying surfaces of the bell and spigot, and any fillet weld defects. The provision of brittle steel, and the further cold working of the bell during fabrication, caused the joints to fail catastrophically in a brittle manner when under tension rather than allowing local yielding at the points of highest stress concentration. Alternatively, the bells may not have failed if steps were taken during joint design and installation to minimize the axial thermal tension and concentration of stress at the toe of the internal fillet weld.

While it is not possible to quantify the relative importance of each potential mechanism of failure of the Atlanta 72-inch WSP, the lessons that the steel water pipe industry can learn from the history of this WSP are clear. The incorporation of these lessons into the appropriate AWWA publications will reduce the risk of joint failure for the future pipeline owner.

- Notch toughness is a critical property of a steel pipeline regardless of the wall thickness. Notch toughness is ensured by specifying a minimum standard for the finished pipeline, or by specifying a quality base steel and limiting cold working during pipe fabrication.
- The design processes for bell and spigot lap joints for WSP should be investigated, quantified, and then standardized in AWWA M11. Joint selection should result from a quantitative analysis of stress and strain through the load path rather than prior industry practice alone—even if this historical experience has been generally positive.
- Control of thermal stress in a restrained cylindrical pressure vessel is basic in principle; however, the practice of controlling pipe installation temperature and designing stress relief capabilities would benefit from explicit documentation of industry best practice.

References

Brockenbrough, R. L. (1990). “Strength of Bell and Spigot Joints.” J. of Structural Engineering, ASCE, 116(7), 1983-1991.

Hunt, L., Jacob, B., Williams, S., Marino, M., and Duppong, J. (2007). “Condition Assessment and Rehabilitation Recommendations to Renew Raw Water Transmission Pipelines for the City of Atlanta.” Conference Proceedings for 2007 ASCE Pipelines Conference, Boston, Mass., July, 2007.

Eberhardt, A. (1990). “108-in. Diameter Steel Water Conduit Failure and Assessment of AWWA Practice.” J. of Performance of Constructed Facilities, ASCE, 4(1), 30-49.

Metallurgical Engineers of Atlanta, Inc. (1982). “Failure Investigation of Third Failure of the 72-inch Steel Water Main.” Prepared for City of Atlanta Bureau of Water, Engineering Division.

Phillips, R. V., Triay, R. Jr., and Marynick, S. M. (1972). “Pipeline Problems—Brittle Fracture, Joint Stresses, and Welding.” J. AWWA, Water Technology/Distribution, 64(7), 421-429.

Wiedeman and Singleton Engineers (1972). “Contract, Bonds and Specifications for Constructing 72” River Water Supply Line for the Hemphill Water Treatment Plant of the Department of Water Works of The City of Atlanta, Georgia.” Nov. 1972.

PCCP Sewerage Force Main Structural Condition Assessment and Asset Management Approach

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INTRODUCTION

The Greater Lawrence Sanitary District (GLSD) learned in 2002 that Prestressed Concrete Cylinder Pipe (PCCP) supplied to GLSD in 1974, was similar to pipelines experiencing a high rate of catastrophic failures throughout the United States. The District began an assessment program in 2005 to assess the structural condition of one of its PCCP lines. This paper discusses the approach GLSD used to assess the structural condition of the pipeline, including finite element analysis, acoustic and transient pressure monitoring, electromagnetic inspection, visual and sounding inspection, and material testing. This paper also discusses the emergency structural repair that was accomplished when one pipe section was found to be in imminent danger of failure.

GLSD treats wastewater from Lawrence, Methuen, Andover and North Andover Massachusetts and Salem New Hampshire. The design capacity of the GLSD treatment plant is 52 MGD. During wet weather, the flow is increased to 110 MGD. Current plans involve raising the peak wet weather capacity of the plant to 135 MGD and in the future to 165 MGD.

GLSD owns over 4 miles of PCCP ranging from 48 to 102 inches in diameter, which was manufactured by Interpace Corporation in 1973 and 1974. Much of this pipe is used in lower pressure applications. However, one GLSD pipeline, the Riverside Pump Station (RPS) 2700 foot long 72 inch diameter force main operates at higher pressures. Virtually all sewage received by GLSD is pumped to the treatment plant through the RPS force main. The force main route is through a residential neighborhood and crosses under an active railroad track that is used by Amtrak, commuter trains and freight trains. The majority of the force main is between 200 feet and 1000 feet from the Merrimack River, a major recreational, water supply and fisheries resource. Failure of this pipeline would have significant financial, environmental and operational consequences.

The GLSD learned of the issues with its Interpace Class IV PCCP in June of 2002 while engaged in planning for combined sewer overflow abatement. In March 2003, the Middlesex County Utility Authority (MCUA) in New Jersey experienced a catastrophic failure of a similar 102 inch PCCP sewer force main. Representatives of the GLSD subsequently visited MCUA to learn about MCUA's response to the failure. MCUA's response included instituting a PCCP condition assessment program. Based on the uncertain structural condition of the GLSD force main, the District, in 2004 determined that it would be prudent to begin a program which would allow it to assess the current structural condition of its 72 inch PCCP force main. GLSD advertised and received proposals from engineering consultants and began its condition assessment program in 2005. GLSD hired CDM to lead the assessment program and they engaged specialty sub-contractors Openaka Corporation and Pure Technologies to conduct the condition assessment program.

GLSD TECHNIQUES FOR ASSESSING THE FORCE MAIN

To assess the structural condition of the force main, the District employed the following analyses/inspection techniques:

Two Dimensional Finite Element Analyses

All the pipe segment designs were analyzed by Openaka utilizing the "Cubic Parabola Design Method" of the American Water Works Association C301-72, Standard for Prestressed Concrete Pressure Pipe, Steel Cylinder Type, for Water and Other Liquids, Appendix A. This standard was used since the manufacturer used it to design the pipe in 1977 and Openaka wanted to check the manufacturer's design. A two-dimensional finite element model was developed to assist in determining the effects of wire breaks on the PCCP. The analysis is assisting GLSD with understanding the structural soundness of the pipe based on the internal pressures within the force main.

Visual and Sounding Inspection

A visual internal inspection involved taking the pipeline out of service and traversing the pipe to visually inspect for signs of deterioration. In addition to the visual inspection, soundings were performed by impacting the pipe wall with a hammer or steel bar to identify areas that sound hollow. This is an important component of an internal inspection as it provides complimentary data to the electromagnetic inspection. Visual and sounding inspections are a reliable means to identify signs of a loss of prestressing in pipe. When found, these pipe sections are usually close to failure. The visual and sounding inspection was performed by Openaka Corporation.

Electromagnetic Inspection

An internal electromagnetic inspection was performed by Pure Technologies to test the continuity of the prestressing wire in the pipeline. A common analogy to describe the physics of electromagnetic inspections is to view the prestressing wire as an electrical coil or inductor. Electromagnetic inspection equipment consists of transmitter and receiver coils that interact with the prestressing wire. As an inductor, the prestressing wire can alter an induced field. The coils are placed in a horizontal configuration with the transmitting coil on one side of the pipe and the receiver coil on the other side. The transmitting coil generates a signal and if the prestressing wire is intact, the receiver coil detects a signal with certain characteristics. When the end of the pipe is reached, the polarity of the detected field reverses because the coiled inductor ends, and then begins again in the next pipe. However, if the inductor is broken (i.e. the prestressing wire is broken), the signal is altered because a new pole reversal occurs part way through the pipe. These unexpected reversals delay the arrival of the signal at the receiver and are quantified to estimate the number of broken wires.

An important aspect of an electromagnetic inspection is to obtain a proper calibration for the type and size of pipe being inspected. This improves accuracy of the results. GLSD was decommissioning and removing a few pipe sections of the force main as part of a treatment plant upgrade. A portion of the pipe sections that were removed were used to calibrate the electromagnetic survey results.

Acoustic Monitoring

An acoustic monitoring system was installed on the force main to continuously monitor the pipeline to detect and locate wire breaks in the prestressing wire wrapping. Wire breaks produce a characteristic noise that can be detected and identified as a break. An underwater hydrophone and six surface-mounted sensors were installed on the pipeline. Using a wireless transmitter, the hydrophones/sensors were connected to a data acquisition system that captures high quality data from acoustic events that meet the basic criteria for classification as a wire failure. Trained signal processors evaluate this data to identify and locate wire failures that occur while the monitoring system is in place. The system was purchased by GLSD to provide a permanent system to continuously track the condition of a pipeline. Acoustic monitoring is being performed by Pure Technologies.

Transient Pressure Monitoring

As a means to better understand what was happening with the force main, GLSD installed a transient pressure monitoring system that could detect the presence of a pressure transient in the pipeline. Under normal conditions the pressure monitoring system samples pressure data once per minute. However, it continuously monitors for pressure transients (e.g. water hammer) or negative pressures, and samples up to once per 0.01 seconds when these events occur. The transient pressure system was supplied by Pipetech International.

Material Testing

Soil, ground water and mortar samples of the exterior coating of the PCCP were taken to determine if environmental conditions were detrimental to the structural condition of the PCCP. In addition, utilizing the decommissioned pipe sections two pipe sections were dissected and mortar and reinforcing wire were retrieved for testing.

CONDITION ASSESSMENT RESULTS

The evaluation of the condition of the 72" PCCP force main began in March 2005 with the installation of a permanent acoustic monitoring and transient pressure systems. An internal inspection and an initial electromagnetic inspection were completed on September 13, 2005. Soil, groundwater, mortar coating and prestressing wire sampling were taken in November and December of 2005. A final pipeline condition report was prepared by Openaka and received by GLSD in November 2006.

Two Dimensional Finite Element Analyses

The finite element analysis (FEA) was to quantify the effect of wire breaks on the integrity of the pipes under service conditions. Internal pressures of 24 psi, 50 psi and 93 psi were examined, representing the prevalent working pressure, average surge pressure and maximum surge pressure based on a sewerage pump shutdown in a power failure. In each case, the analysis determined the number of wire breaks required to exceed the concrete tensile strength of 470 psi which would result in the cracking of the concrete core.

The FEA results showed that at 24 psi, the concrete core would not crack regardless of the number of wire breaks and the circumferential concrete tensile stress plateaus at only around 270 psi. At the average surge pressure of 50 psi, the concrete tensile strength of 470 psi is exceeded with 40 contiguous prestressing wire breaks. At the maximum surge pressure of 93 psi, the concrete tensile strength of 470 psi is exceeded with 20 contiguous prestressing wires breaks.

Visual and Sounding Inspection

The interior of the pipeline was visually inspected for cracks, spalls or other signs of distress, while also being sounded for lining separations. The location of pipe joints and other appurtenances, such as valves and manholes, were determined by the use of a distance wheel to allow verification of the as-built conditions of the pipeline.

The measurements determined that the manufactures lay length data did not reflect the as-built conditions. A 4' short, pipe section 133, was found to be hollow over its entire length. It is suspected that the short may be a steel special, however, it can not be ruled out as potentially distressed PCCP without further investigation. Also, a 20' pipe, pipe section 22, was found to have a hollow area at the spring-line 22 inches longitudinally and 33 inches circumferentially located approximately 45 inches from the pipe joint. Pipe section 22 was repaired under an emergency procurement which is discussed in further detail later in this paper.

Acoustic Monitoring

The acoustic monitoring system has been reporting the number and location of wire break information since March 17, 2005, identifying wire breaks under two classifications:

Class A Wire Breaks- Match all acoustic criteria for wire breaks. These events are almost certainly caused by wire breaks.

Class B Wire Breaks- Match most of the important acoustic criteria for wire breaks. These events are most likely wire breaks.

As of November 21, 2006, a total of 101 wire breaks have been detected, 21 Class "A" wire breaks, 65 Class "B" wire breaks and 15 of which the type is unknown. The wire breaks detected by pipe section number are presented in Table 1. The pipe section noted as "Upstream Pipes" in Table 1 includes a 6 foot long section, a 2 foot long short and a 20 foot long section that are located upstream of the first acoustic sensor and therefore wire break type and location can not be determined. These breaks are noted in Table 1 as unknown.

TABLE 1
WIRE BREAKS BY PIPE CLASS

Pipe Number	**Pipe Length**	**Total Breaks**	**Class "A" Breaks**	**Class "B" Breaks**
Upstream Pipes	Unknown	15	Unknown	Unknown
3	20	1	0	1
4	20	2	0	2
15	18	1	0	1
16	19	32	0	32
17	18	1	0	1
21	20	6	1	5
22	20	17	8	9
23	8	3	0	3
25	20	1	1	0
26	20	1	0	1
36	20	6	0	6
37	8	1	0	1
62	20	1	1	0
63	20	1	0	1
83	20	1	0	1
87	20	1	1	0
88	20	1	1	0
89	20	3	3	0
105	19	1	1	0
108	20	1	0	1
114	20	1	1	0
129	20	1	1	0
130	20	1	1	0
131	20	1	1	0
Totals		101	21	65

To date, the acoustic monitoring system has alerted the District to three pipe sections of concern; the upstream pipe sections, pipe section 16 and pipe section 22.

Pipe section 16 is a 19 foot long pipe. The majority of wire breaks were recorded within this pipe section after the force main was re-pressurized after being dewatered for the internal inspection and the electromagnetic inspection. This pipe section has had a total of 32 wire breaks, with 28 wire breaks taking place over a 3 month period. The 28 wire breaks were clustered within seven feet of each other and began at approximately 4 feet from the downstream end of the pipe joint.

Pipe section 22 is a 20 foot long pipe section and exhibited 13 wire breaks during the first six months of acoustic monitoring. Based on this early information, GLSD determined that the pipeline should be dewatered to allow an internal inspection and electromagnetic evaluation to be performed to ensure the safety of the pipeline.

Electromagnetic Inspection

The majority of the pipe sections inspected did not exhibit electromagnetic anomalies consistent with wire break damage. However, 14 out of the 160 pipe sections, 9% of the total, had electromagnetic anomalies consistent with wire breaks. Four of the pipe sections (Pipes 22, 89, 105 and 131) have electromagnetic anomalies consistent with wire break damage of 50 or more wire breaks. Table 2 below presents the electromagnetic and the acoustic results for those pipe sections:

TABLE 2
Acoustic Wire Break & Electromagnetic Inspection Results

Pipe Section	Pipe Length (feet)	Estimated Wire Break Damage Based on Electromagnetic Inspection (after calibration)	Number of Wire Breaks Detected by Acoustic Monitoring System
142	20	1	0
131	20	60	1
130	20	10	1
126	20	10	0
119	20	15	0
105	19	100	1
95	20	5	0
89	20	80	2
49	20	20	0
22	20	80	17
21	20	30	6
17	18	40	1
7	20	25	0
3	20	14	1

Utilizing the decommissioned pipe sections, two pipe sections were dissected to confirm the actual number of existing wire breaks. One pipe section had a dented steel casing and the other pipe section was found to have a reinforcing splice. The electromagnetic evaluation predicted that both pipe sections had a number of wire breaks. It appears that both of these pipe conditions attributed to the electromagnetic evaluation determining that there were electromagnetic anomalies found.

Actions Based on Inspection and Data Assessment

During the internal soundings, pipe section 22 was found to have a large hollow area. Additionally, the electromagnetic inspection predicted this pipe section had 80 wire breaks. This pipe section was therefore determined to be in imminent danger of failure and GLSD implemented emergency repairs to pipe section 22, utilizing post tensioning cables, as discussed later in this paper.

Transient Pressure Monitoring

Normal operating pressures within the force main are approximately 24 psi. There are no valves within the force main system and it was thought that there were no major pressure surges within the system. The PCCP was designed for a working pressure of 40 psi and a working plus surge pressure rating of 80 psi. After the transient pressure system was installed, it was found that the force main was subject to excessive transient surges from pump operations. One pressure surge was recorded during a power failure at 92 psi. These realizations lead the GLSD to prioritize the rehabilitation of the 28 year old surge control valves on each pump to control any water hammer surges. Rehabilitation of the surge control valves and careful operation of the pumps have reduced the magnitude of pressure surges. Controlling large transient surges is important to the integrity of the pipe since these surges could stress the pipe and contribute to wire breaks and ultimately cracking of the mortar coating.

Material Testing

Samples of soil, groundwater and exterior pipe mortar were taken at different locations along the force main. The soil and ground water samples were tested to determine their aggressivity to the mortar coating and the reinforcing steel. The mortar coating samples were petrographically evaluated to determine their ability to protect the prestressing wires and their ability to resist premature failure. The results of the laboratory tests are summarized below:

Table 3
Laboratory Testing Results

Location	Station	Groundwater	Soil	Mortar Coating	Prestressing Wire
Pipe 21	3+55	N/A*	N/A	Good	N/A
Pipe 22	3+68	Aggressive	Aggressive	Poor	N/A
Pipe 68	11+08	N/A	Neutral	Fair	N/A
Pipe 130	23+37	N/A	Neutral	N/A	N/A
Pipe 142		N/A	N/A	Good	H.E. Sensitive**
Pipe 143	24+97	N/A	Neutral	Good	H.E. Sensitive
Pipe 144	25+18	N/A	Neutral	Good	N/A

Pipe 147		N/A	N/A	Fair	N/A

* N/A refers to no samples available for testing ** H.E. Sensitive refers to hydrogen embrittlement sensitivity

The results of the laboratory testing determined that pipe section 22 was failing as a result of a poor mortar coating that was not protecting the pipe from the aggressive environment surrounding it. It was unclear if the poor coating was the result of a mortar defect or the aggressive nature of the surrounding environment. Although pipe section 68 had a mortar coating of marginal quality, the soil surrounding the pipes was not aggressive. Therefore, this pipe was in better condition. In fact, only the soil collected at pipe section 22 appeared to have conditions detrimental to PCCP as both the soil and groundwater were found to be aggressive. The remaining mortar coating samples were of good quality, and the remainder of the soil and groundwater samples was found to be not aggressive.

Prestressing wire from pipe sections 142 and 143 were tested. The results from the metallurgical testing of the 6 gage Class IV was:

1. Tensile properties of the tested wire samples were found to be generally higher than the minimum tensile requirements specified by the manufacturer.
2. Torsion test results confirmed the presence of a high degree of dynamic strain aging in the wire.
3. Hydrogen embrittlement (HE) sensitivity testing employing the FIP ammonium thiocyanate test confirms a “somewhat severe” to “severe” HE sensitivity.
4. The combined mechanical property/hydrogen embrittlement test results indicate that the wire is dynamically strain aged and susceptible to an embrittlement mode of failure if corrosion were to initiate on the wire. Due to this degree of HE sensitivity, however, the pipe is not recommended as a candidate for impressed current cathodic protection. Alternatively, if properly bonded and jumped for continuity, the prestressing wire could be safely protect from corrosion by installation of a sacrificial zinc anode cathodic protection system.

GLSD’S Pipe Section Strengthening/Repair Project

After receiving the results of the acoustic, electro-magnetic and visual inspection reports for pipe section 22, the GLSD Board of Commissioners voted that this pipe repair should proceed under an emergency procurement procedure. As required by state law, the GLSD applied for and received an Emergency Waiver from the Commonwealth of Massachusetts’ Division of Capital Asset Management. The declaration allowed the normal bidding procedures to be dispensed with. Four contractors that were previously pre-qualified during preparation of a Emergency Response Plan were requested to provide bids.

The emergency procurement declaration was issued on October 13, 2005. The construction contract was awarded on November 15, 2005, preparatory construction work began on November 21, 2005, and actual excavation began on November 28, 2005. Working three straight 24 hour periods, the contractor excavated the pipe, performed the pipe strengthening and backfilled the trench while the pipeline was in operation. Restoration of the site was completed on December 7, 2005.

The District self-performed a modified construction management project delivery approach. GLSD procured the repair tendons, managed and coordinated work of the various utilities, the prime engineering consultant and its PCCP sub-consultant, and hired and directed the work of the contractor (R.H. White Construction Company) under a time and materials approach.

The pipe strengthening procedure utilized post tensioning tendons installed on the exterior mortar coating of the entire 20 foot length of pipe section 22. The tendons used were 0.6 inch seven strand wire cables encased in a plastic sheath and spaced with six-inch spacing along the pipe. The tendons were supplied by DSI Lang and were specified under ASTM A416, Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete, to be 270 ksi strand with a Guaranteed Ultimate Tensile Strength of 58 ksi. The tendon was encased in an extruded polypropylene sheath 80 mils thick. Voids between the polypropylene sheath and the tendon were filled with a corrosion inhibiting grease. The grease helped to reduce the friction between the strand and the sheathing, which allowed for the maximum transmission of the post-tensioning force over the undisturbed mortar coating.

Anchorage of the tendons was accomplished with DSI Lang's proprietary lokcoupler and three piece wedge anchors. The lokcouplers were epoxy coated and provided with zirk fittings to inject corrosion inhibiting grease after tensioning. A special hydraulic stressing device was used to stress the stand to the specified load of 44 kips. The hydraulic stressing unit applied the required tendon load and then the split wedge anchor was set to secure the tendon. This process was repeated until the entire length of pipe was tendon reinforced. The excess ends of the tendons were cut approximately two to three inches from the split anchor wedges with an abrasive cut-off wheel and then capped with a rubber boot. The steel couplers and the ends of the tendons were then encased in concrete to provide further protection from corrosion. This strengthening/repair method was chosen as the preferred alternative because the tendons could be installed while the force main remained in operation.

The construction cost to strengthen/repair this 20 foot pipe section was approximately $142,000 not including consulting and engineering services, or GLSD personnel's time managing and coordinating the work.

CONCLUSIONS FOR PIPELINE MANAGEMENT

The evaluation of all of the internal and external inspections as well as the laboratory testing results of the samples collected from the Greater Lawrence Sanitary Districts 72-Inch Sewage Force Main lead to the following general conclusions:

Conclusions

1. For the 72-inch diameter PCCP operating at the average working pressure of 24 psi, the concrete core is not predicted to crack. At around 50 contiguous wire breaks, the tensile concrete stress plateaus at around 270 psi. At the average surge pressure of 50 psi, the 72-inch PCCP requires 40 contiguous wire breaks before the concrete core is predicted to crack and accelerated environmental corrosion can be expected.
2. The steel cylinder, if the effects of the external loading are ignored and if the assumption that the core has not cracked to the point to allow the onset of corrosion, is capable of handling up to 47 psi of internal pressure without failing.
3. The hollow concrete lining of the 4-foot short pipe, Pipe section 133 (Station 23+81 to Station 23+85), could potentially represent distress if the pipe is PCCP and not concrete lined steel pipe. Since no potential wire breaks were found in the preliminary P-Wave electromagnetic testing report and no wire break events have been detected through acoustic monitoring, the hollow lining of this short section does not appear to be due to prestressing wire breakage.
4. The pipes were manufactured with "Class IV" wires which had a severe sensitivity to hydrogen embrittlement and a high degree of dynamic strain aging. The wires would be susceptible to failing if exposed to minimal corrosion.
5. The test results from the soil and groundwater collected from Pipe section 22 (Station 3+51 to Station 3+71) at Station 3+68 indicated conditions detrimental to the pipe. The samples of the soil collected at the remaining locations were not found to be aggressive.
6. The mortar coating sample collected from Pipe section 22 was of poor quality and not adequately protecting the prestressing wires. The hollow area on the interior and hollows with longitudinal cracking on the exterior of the pipe were most likely the result of the combination of environmental deterioration attributable to the high porosity mortar coating allowing corrosion and subsequent hydrogen embrittlement failure of the Interpace Class IV prestressing wires.
7. The electromagnetic survey predicted 60 or more broken wires in Pipe section 89 (Station 15+09 to Station 15+29), Pipe section 105 (Station 18+28 to Station 18+47) and Pipe section 131 (Station 23+41 to 23+61). The number of broken wires on these pipes exceed the limit at which the core will crack based on the finite element analysis if the pipe experiences an internal pressure of 50 psi.
8. The repair of Pipe section 22 (Station 3+51 to Station 3+71) using seven strand tendons proved that the pipe could be repaired without removing the pipeline from service as long as the internal pressure of the pipeline can be maintained at a low and constant pressure.
9. Two areas have experienced significant wire breaks during the acoustic monitoring. The portion of the pipeline upstream of the first monitoring station and Pipe section 16 (Station 2+65 to Station 2+85) are the only areas that have experienced significant wire breaks. The rate of wire breakage in Pipe section 16 over a short period of time may be the result of advanced structural deterioration.

Recommendations

At this time, the GLSD Board of Commissioners is reviewing the final conditions report and its conclusions and recommendations.

Condition Assessment of Prestressed Concrete Cylinder Pipe used in the Circulating Water System at Great River Energy's Coal Creek Station

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Abstract

Prestressed concrete cylinder pipe (PCCP) is often utilized in large volume circulating water systems for energy generation facilities. The circulating water system is essential to plant operations, and for many facilities, redundancy may not have been built-in; thus failure of a section of non-redundant PCCP could be disastrous. Evaluation of the physical condition of the circulating water system and assessment of the potential operational risk is deemed good engineering practice.

At Great River Energy's (GRE) Coal Creek Station (CCS), a two unit 1100 megawatt pulverized coal generation facility, generation leaders worked with Golder Associates (Golder) to perform a condition assessment of the PCCP utilized in their circulating water system. This condition assessment involved the following: establishment of goals and constraints; review of documentation relating to pipe manufacture, installation, and operations; development of work plan; physical observations including visual, sounding, and remote field eddy current / transformer coupling (RFEC/TC); repairs including grout repair, joint seals, and exterior

structural bands; condition summary of the existing system; and development of recommendations for future evaluations, repairs, and/or replacement.

The PCCPs evaluated at CCS included a 36-inch lined cylinder pipe, and both a 96-inch and 120-inch embedded cylinder pipe manufactured by Interpace Corporation utilizing class III and class IV prestressing wire. Installation occurred in 1976 and the system has been online without a double unit outage since 1979. The system conveys approximately 345,000 gpm of cooling water and is founded in clayey soils with groundwater containing high levels of sulfates and chlorides. Numerous electrical duct banks parallel and cross the pipeline, as well as crossing overhead transmission lines and areas with significant stray current.

Document review and risk assessment of the pipeline prompted scheduling of physical observations and development of possible repair options that could occur during the first planned double outage in over twenty-seven years of operations at CCS. This outage provided approximately 48 hours in which the pipeline could be physically evaluated and repairs effected. A strict requirement was not to postpone plant start-up unless imminent pipeline failure was suspected.

The physical evaluation demonstrated a strong correlation between visual observations, sounding and RFEC/TC results. A section of the pipeline running adjacent to the cooling water towers and an electrical duct bank was found to have several distressed pipe sections. Limited observations of the prestressing strands showed minimal visual corrosion with most wires exhibiting clean breaks reflective of hydrogen embrittlement, possibly due to stray currents. Select repairs were completed during the outage and a formal plan was developed for future evaluations, repair of key areas, and the design of a replacement line for severely distressed pipeline sections.

Background

Great River Energy's (GRE's) Coal Creek Station (CCS) is North Dakota's largest coal-fired power plant, with two-units generating 1180 MW of power, serving 1.3 million people. Construction began in 1974 with Unit 1 coming online in 1979 and Unit 2 coming online in 1981. A critical component of the power plant is the common circulating water system. Water is pumped from a cooling basin to the condensers by a combination of four centrifugal pumps (located in the pump house) through the 120-inch/96-inch (305cm/244cm) diameter prestressed concrete cylinder pipe (PCCP) supply line (~1000ft/305m). Steam generated by the turbine generator is condensed by the cooling water within the condensers. The heated cooling water is then sent to the cooling water towers through the 120-inch/96-inch (305cm/244cm) diameter PCCP return line (~2000ft/610m) and sent from the cooling towers to the cooling basin. In addition, the system also contains a 36-inch/24-inch (91cm/61cm) diameter PCCP auxiliary cooling water line (~1000ft/305m) that runs from the pump house to the plant auxiliary cooling heat exchangers. The heated auxiliary cooling water discharges into the circulating water return line to the cooling towers. The piping configuration of the circulating water system is illustrated in Figure 1.

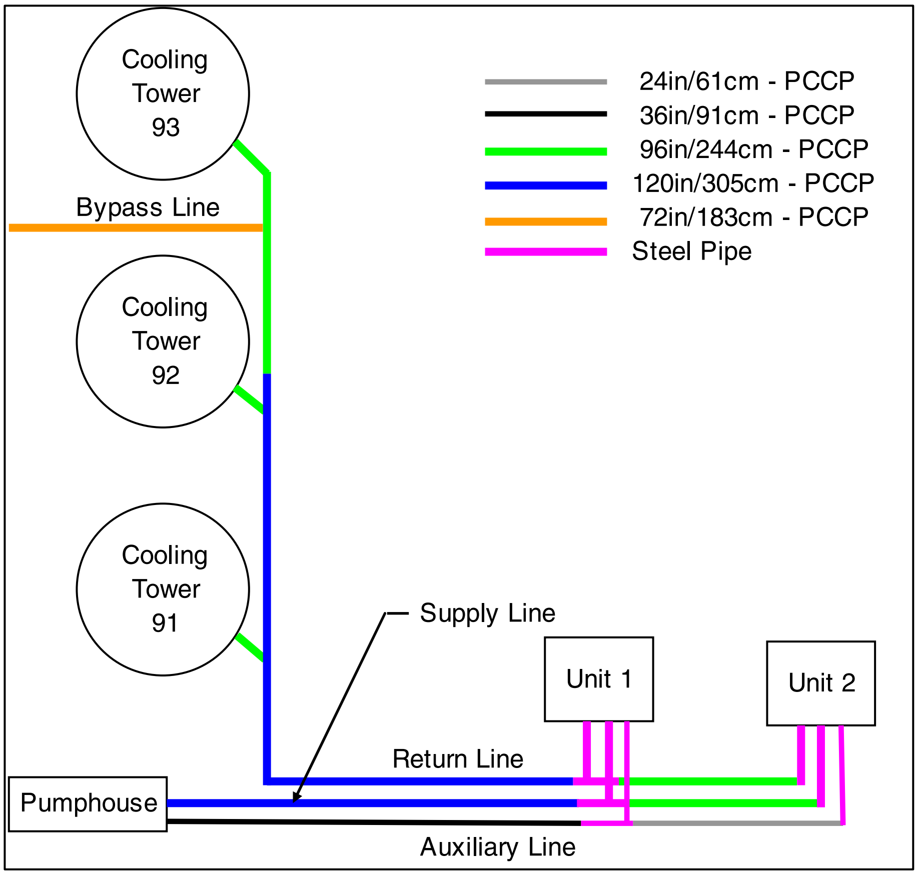

Figure 1 –Circulating water system

The circulating water system is common to both boiler Units, and has been in service since 1979 without an outage due to the historic infeasibility of a dual-unit outage. Recent changes to GRE's distribution agreements made a 5-day dual-unit outage economically feasible, and a dual-unit outage was planned for May 2006. GRE generation leaders worked with Golder Associates Inc. (Golder) and Brunzell Associates (Brunzell) to perform a condition assessment of the PCCP utilized in the circulating water system coinciding with the planned dual-unit outage.

Condition Assessment

The condition assessment performed for the PCCP used in the circulating water system at CCS involved the following activities: establishment of goals and restraints; document review; physical evaluation and repairs; condition summary; and development of recommendations.

Goals and Constraints

Goals for the condition assessment of the circulating water system were developed by GRE and Golder personnel during various planning meetings. The goals of the condition assessment were to: evaluate the current condition of the pipe; effect repairs, as possible, during the outage; evaluate the remaining service life of the

PCCP; and develop recommendations with respect to future condition assessment, repairs, and possible replacement of the circulating water system pipe.

Constraints for the condition assessment included: limited information on the installation of the circulating water system pipe; strict time requirements for the physical evaluation and repairs based on the dual-outage schedule (48-60 hours); and health and safety and access limitations.

Document Review

Document evaluation included review of the design basis for the PCCP component of the circulating water system, construction information, operational information, industry information, and previous evaluations and repairs.

The document review indicated that the PCCP may be susceptible to distress based on the following findings:

- The PCCP was Lock Joint Pipe manufactured in 1975 by Interpace Corporation, which has gone bankrupt due to failure of large diameter PCCP.
- The majority of the PCCP was manufactured with Class IV prestressing wire, which may be prone to hydrogen embrittlement.
- The groundwater and circulating water have high levels of chlorides and sulfides, which may lead to corrosion of the prestressing wires and steel cylinder if mortar coating is compromised.
- The locations of parallel and perpendicular power lines may impose stray currents on the pipelines, leading to embrittlement of the prestressing wires.
- No cathodic protection was clearly specified or implemented in the manufacture and installation of the circulating water system.
- Leaking joints previously repaired within the auxiliary cooling water pipeline may indicate improper bedding and backfill, or improper joint installation.

Physical Evaluation

The physical evaluation took place during the planned dual-outage between May 16, 2006, and May 20, 2006. The evaluation included an interior electromagnetic survey, interior visual observation and pipe sounding, exterior visual observation and pipe sounding at selected locations. As a result of the physical evaluation, repairs were identified. These repairs included mortar replacement at joints and placement of Weko-Seals at joints, and external repairs included the installation of steel support bands on severely distressed pipes. Additionally, ports and accelerometers were installed at select locations to allow future acoustic emission testing, if needed.

Health and safety planning for the physical evaluation was a significant task focusing on excavation and confined space entry.

Electromagnetic Survey

An electromagnetic survey using remote field eddy current/transformer coupling technology was performed by the Pressure Pipe Inspection Company (PPIC). PPIC utilized a manned inspection tool to survey the supply line and return line, and used an unmanned remote crawler inspection tool to survey the auxiliary cooling line.

PPIC surveyed approximately 3,665 feet (1,117m) of PCCP consisting of 201 individual pipe sections. Of the 201 pipe sections surveyed in the circulating water system, PPIC identified 43 pipe sections that showed clear indications of distress: three pipe sections in the supply line, 38 pipe sections in the return line and two pipe sections in the auxiliary cooling water line (Table 1).

Table 1 - Summary of Electromagnetic Survey Findings

Line	Diameter (in/cm)	# Inspected Pipe Sections	Length (ft/m)	Distressed Pipe Sections	PCCP with <25 WB	PCCP with 25 to 50 WB	PCCP with >50 WB
Supply	96/244	14	277/84	2	2	0	0
	120/305	39	763/233	1	0	1	0
Return	96/244	35	638/194	19	9	6	4
	120/305	67	1273/388	19	12	4	3
Aux.	36/91	46	714/218	2	2	0	0
Total		201	3665/1117	43	25	11	7
Percentage				21.4%	12.4%	5.5%	3.5%

Internal/External Visual/Sounding

Internal visual observations and pipe sounding were performed on the supply line and return line, but were not performed on the auxiliary cooling line or the 72-inch (183cm) bypass line due to access and isolation restraints. Visual observations included looking for loose or missing joint mortar, leaking joints, gouges, circumferential cracks, diagonal cracks, and longitudinal cracks. Pipe sounding was performed by tapping a steel rod with end caps against the pipe wall around its circumference while walking down the pipe. Approximately 2,950 feet (899m) of PCCP consisting of 155 pipe sections were visually observed and sounded internally. Only three of the pipe sections observed showed indications of significant distress.

External visual observations and pipe sounding were performed at four excavations chosen based on potential risk factors, accessibility during the outage, potential locations for installation of acoustic monitoring instrumentation, and preliminary findings from internal observations and the electromagnetic survey. Visual observations included looking for cracks in any direction, the condition of diapered joints, rust stains and chemical precipitate buildup at cracks, and signs of corrosion/breakage on any exposed prestressing wires. External pipe sounding was done with a one-pound hammer being tapped against the pipe wall around the exposed circumference. Approximately 12 PCCP pipe sections were visually observed and sounded externally. Only three of the pipe sections observed had any indications of distress.

Noted observations included:

- The supply line had no visual signs of structural distress (no hollows, diagonal cracks, or longitudinal cracks).
- No leaking joints were observed in either the supply line or return lines were found. Some joints appeared to have potential for leakage and were marked for repair (particularly steel to PCCP transitions).
- Both the supply line and return lines had internal circumferential cracks near the spigot end and mid-barrel. The 96-inch (244cm) portion of the return line had the most internal circumferential cracks.
- An internal diagonal crack in the concrete was found on pipe R103 in the east-west portion of the return line. No hollows were found around this crack. External observations of this pipe found no cracks or hollows. The electromagnetic survey did not estimate any wire breaks for this pipe.
- External longitudinal cracks and hollows were found in the first six feet of R332 (north-south portion of the return line). A small window of mortar coating was broken off to reveal prestressing wires. Several broken wires were identified with no significant corrosion present, and no necking down at the point of failure. Based on the break observations, the prestressing wire failure is likely due to hydrogen embrittlement from stray currents. No cracks in the concrete or hollow areas were found during the internal observation of R332. The electromagnetic survey estimated 120 wire breaks in pipe R332. (Figure 2)

Figure 2 - Broken prestressing wires and external longitudinal crack on pipe R332

- An internal longitudinal crack in the concrete was found along the length of pipe section R329 (north-south portion of the return line). A circumferential crack crossing the longitudinal crack near the middle of the pipe section had a hollow spot. External longitudinal cracks and hollows in the concrete were found running across two-thirds of pipe R329. The electromagnetic survey estimated 230 wire breaks in pipe R329. (Figure 3)

Figure 3 - Longitudinal cracks on pipe R329 (internal and external)

- A five-foot long internal longitudinal crack in the concrete was found on pipe R345 (north-south portion of the return line). No hollows were found around this crack. An external longitudinal crack and hollow in the concrete was observed running across two-thirds of pipe R345. The electromagnetic survey estimated 205 wire breaks in R345. (Figure 4)

Figure 4 - Longitudinal cracks on pipe R345 (internal and external)

Repairs

Due to the short duration of the dual-outage, limited repair options were available. However, it was deemed prudent to effect any repairs possible during the outage. Anticipated repairs included installation of Weko-Seals at joints, mortar repair at joints, and the installation of exterior steel support bands.

During the visual observations, joints with soft mortar were identified, all loose mortar was removed until competent mortar or the steel joint was found, and the joint was filled with SikaRepair 223 mortar flush with the inside surface of the pipe. A total of 40 joints were identified which received some mortar repair.

Weko-Seals were installed on joints showing potential for movement and/or leakage. Seven joints were identified for Weko-Seal installation (Figure 5).

Figure 5 - Weko-Seal installed at steel/PCCP joint (before and after)

Steel support bands for the 120-inch (305cm) and 96-inch (244cm) PCCP were manufactured by Price Brothers for potential use during the dual-outage. The steel bands were ¼-inch (6.35mm) thick by six-inch (152mm) wide steel bands that matched the outside diameter of the pipes. Results from the physical evaluation identified three pipe sections in distress that warranted installation of these steel bands (Pipe sections R329, R332, and R345). The steel bands act to support the tension capacity of the remaining prestressing wires and to hold the concrete coating in place. The steel bands are a temporary structural support and are not a permanent repair for these pipes. Prestressing wires may continue to break in these pipes, and cracks in the concrete lining will allow corrosion and eventual failure of the steel cylinder. (Figure 6)

Figure 6 - Steel support band installation

Instrumentation

Ball valves were installed on all the manhole access points along the pipeline and in the cooling towers to allow future insertion of hydrophones and accelerometers were installed on the top of the pipe at three excavations. This will allow acoustic emission testing of the pipeline should it be deemed necessary in the future.

Condition Summary

A strong correlation was observed between hollow-sounding areas, internal longitudinal cracks in the concrete, electromagnetic survey estimated wire breaks, and external longitudinal cracks in the concrete.

Visual observations and pipe sounding identified three pipe sections in the north-south portion of the return line with significant distress (R329, R332, and R345). The signs included internal longitudinal cracks in the concrete, hollow areas on the inner core of the pipe, external longitudinal cracks in the concrete, and a hollow area on the outer core of the pipe.

The electromagnetic survey identified 43 of 201 pipe sections with some level of distress. The estimated number of wire breaks in these pipe sections was five to 230 wire breaks. The north-south portion of the return line has 32 of the 43 pipe sections identified with distress. All of the significantly distressed pipes (>50 wire breaks) identified are located in the north-south portion of the return line.

The three pipe sections identified as significantly distressed by visual observations and pipe sounding were also identified by the electromagnetic survey. These three pipe sections (R329, R332, R345) were temporarily reinforced with external steel bands.

Recent research has shown that PCCP can still function normally with wires that are cut under controlled laboratory conditions. Hydrostatic pressure testing has shown pipes with approximately four-feet (122cm) of wire loss functioning up to a failure pressure of approximately 90 psi (620 kPa) (Zarghamee 2003a). In the north-south portion of the return line at CCS, as many as 230 broken wires are estimated by PPIC in a single barrel of the pipe. This number, if continuous, represents a length of about ten feet in the barrel of the pipe. This number of wire breaks in any configuration, results in a failure of the pipe's integrity and total dependence on the cylinder and block arching to resist internal pressure. The low operating pressure of the return line (~50psi/345kPa) in relationship to its design has allowed these distressed pipe sections to remain in operation despite the estimated number of wire breaks. However, the service life of a pipe in this condition is limited as deterioration of the prestressing wire and corrosion of the steel cylinder wall are progressive.

The electromagnetic survey identified 43 pipes with between five and 230 wire breaks. The lower end of five wire breaks is within the design limitations of the PCCP and no action is required. The upper end of 230 wire breaks is beyond the design limitations of the PCCP and action is required. To understand the action level of wire breaks between these extremes, a structural analysis model of the particular PCCP and loading conditions can be made. Dr. Mehdi Zarghamee has created limit-state curves from these types of structural analysis models (Zarghamee 2003b). The limit-state curves divide the condition of the pipe into different priorities from imminent failure to low risk of failure based on the operating pressure and the number of wire breaks.

General observations of these limit state curves indicate that under low operating pressures, the pipe can undergo a significant number of wire breaks before entering into a priority where failure is expected to occur with time. A pipe-specific structural analysis has not been performed for the PCCP at CCS to determine the threshold number of wire breaks (number of breaks indicating imminent or eventual failure). However, based on the physical evaluation findings, the circulating water system at CCS can be divided into two sections; the north-south portion of the return line and the remaining lines. Within the north-south portion of the return line, three pipe sections have been identified in a state of significant distress, and 6 more pipe sections are estimated to have between 50 and 200 wire breaks suggesting eventual to imminent failure. Within the remaining lines, the highest estimated wire breaks is 35, and a low risk of pipe failure is expected for the few distressed pipe sections outside of the north-south portion of the return line.

Recommendations

Based on the condition assessment performed, the following actions were recommended to GRE:

- Purchase emergency repair sections for the 96-inch (244cm) and 120-inch (305cm) PCCP;
- Replace the north-south portion of the return line;
- Install cathodic protection in the new north-south portion of return line and at select locations of the existing supply line, the east-west portion of the return line, and the auxiliary cooling line;
- Perform future physical evaluation of PCCP to determine rate of distress and identify any new concern areas;
- Do not perform acoustic monitoring unless the repair/replacement of the north-south portion of the return line is not planned to occur within the next few years, or if future constraints will limit the ability for plant outages for a full physical evaluation of the pipeline;
- Repair or replace sections of the supply line, east-west portion of the return line, and auxiliary line as deemed necessary from future physical observations.

References

Zarghamee, M.S. (2003a). "Hydrostatic Pressure Testing of Prestressed Concrete Cylinder Pipe with Broken Wires," *New Pipeline Technologies, Security, and Safety*, Mohammad Najafi, Ed., ASCE, Proceedings of the ASCE International Conference on Pipeline Engineering and Construction in Baltimore, MD, 13-16, Jul., pp. 294-303.

Zarghamee, M.S., Eggers, D.W., Ojdrovic, R, and Rose, B. (2003b). "Risk Analysis of Prestressed Concrete Cylinder Pipe with Broken Wires," *New Pipeline Technologies, Security, and Safety*, Mohammad Najafi, Ed., ASCE, Proceedings of the ASCE International Conference on Pipeline Engineering and Construction in Baltimore, MD, 13-16, Jul., pp. 599-609.

BALTIMORE'S PILOT WATER MAIN INSPECTION PROGRAM BECOMES EMERGENCY REHAB / REPLACEMENT PROJECT

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ABSTRACT

This paper reviews the results of the City of Baltimore's pilot large-diameter pipe inspection program, originally undertaken to develop and implement an inspection, condition assessment and asset management program for raw and finished water mains within the Baltimore Metropolitan Area. The transmission mains of primary concern are those of prestressed concrete cylinder pipe (PCCP), with a particular emphasis on those constructed in the early 1970's. Many of these PCCP transmission mains were fabricated with Class IV prestressing wire and such mains have had a significant history of breaking.

An unexpected outcome of the inspections was the identification of a large number of 54- and 36-inch pipe sections in deteriorating condition which posed the threat of incipient main failure if returned to service. These findings required that the joint owner/operators of the system repair or replace the identified pipe reaches on an emergency basis, while at the same time maintaining adequate water supply for the system's customers. The paper describes the two types of internal inspections that were employed for this project, the action plan developed in response to the defective pipes identified, and the design and construction of repairs and replacements of the damaged pipes. It also discusses the careful coordination required between Baltimore City, and Baltimore, Howard, and Anne Arundel Counties in response to the inspection findings.

PROGRAM BACKGROUND/OVERVIEW

The Baltimore Metropolitan Water System includes approximately 200 miles of transmission mains from 36 to 108 inches in diameter and serves retail customers in both Baltimore City and Baltimore County. It also serves finished water on a wholesale basis to Anne Arundel and Howard Counties which operate their own transmission and distribution systems. Most of the larger water transmission mains in the Metropolitan Water System were constructed using prestressed concrete cylinder pipe (PCCP). Much of the PCCP was constructed in the 1970's with "Class IV" prestressing wires. Such pipe has had a history of failure stemming from wire breakage due to hydrogen embrittlement. Figure 1 shows the various large diameter mains in the system and locations of recent failures.

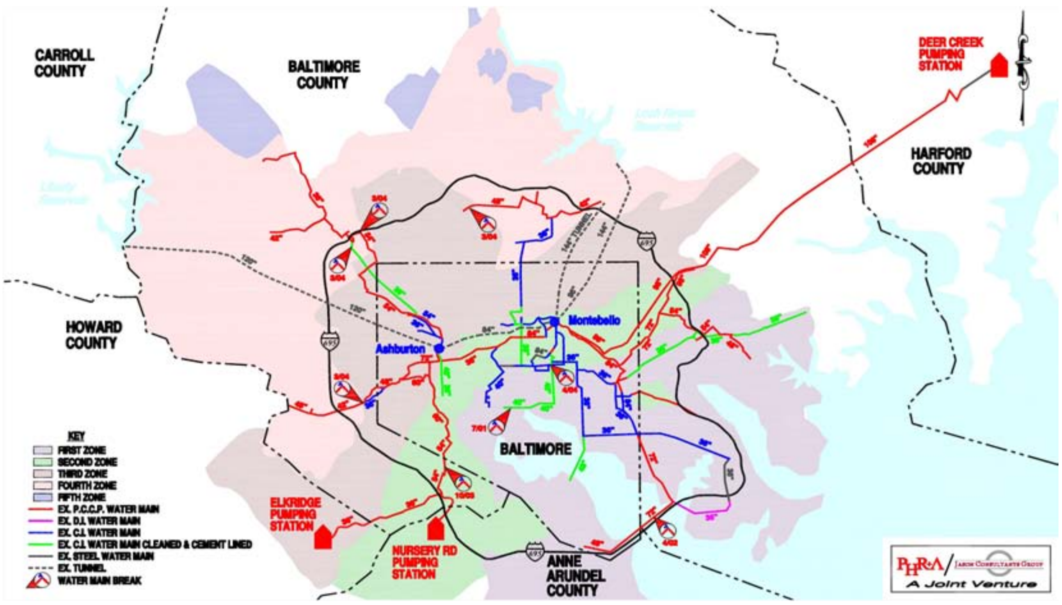

Figure 1. Baltimore Metropolitan Water System Large Diameter Mains

PILOT INSPECTION PROGRAM

Because of its concern about the integrity of the Metro region's water supply, the City, as the primary owner and operator of the system, retained PHR+A / Jason Consultants, A Joint Venture to conduct an assessment of the large diameter pipe system. The project scope included the development of a pilot inspection program for the purpose of developing and implementing an inspection, assessment, preventative maintenance and monitoring program for raw and finished water mains within the Metropolitan Area. The City required that the focus of the work be on PCCP and ferrous pipe with diameters of 36 inches and greater. The emphasis of the inspection program was centered on structural conditions with secondary consideration being given to hydraulic and quality issues. Further, the project scope required that a water main rating system be established for the purpose of prioritizing those reaches requiring inspection and/or monitoring either as part of the initial pilot inspection/monitoring program, or as part of a future inspection/monitoring program.

The major elements of the scope of the pilot program included:

- Selection of water main reaches to be included in pilot program
- Selection of inspection methodology
- Preparation of inspection, dewatering, confined space entry, and traffic control plans
- Performance of pipeline inspection
- Evaluation of inspection results
- Identification of water main reaches requiring emergency repairs and/or replacement

INSPECTION METHODOLOGY

Two internal inspection methodologies were employed ---- visual inspection and sounding, and electromagnetic inspections.

A. Visual Inspection and Sounding

The visual surveys consisted of measuring and checking each pipe for cracks and signs of structural distress. All cracks, circumferential and longitudinal, were noted and photographed. A ½-inch steel bar approximately 6-inches shorter than the inside of the pipe was used for the sounding inspection. Areas of pipe that return a hollow sound when tapped with the bar generally indicate separation between the inner core and outer layers of the pipe. Such separation usually suggests that prestressing wires are broken and the pipe is subject to potential structural failure. Soundings were made at 2-inch intervals between the 1 and 5 o'clock positions, and from 7 to 11 o'clock. The limits of all hollow areas were noted and recorded.

B. Electromagnetic Inspections

Internal electromagnetic inspection is a state-of-the-art technique used to obtain a baseline condition assessment of all the pipe sections in a PCCP main. The inspections rely on passing equipment through the pipe that senses electromagnetic anomalies caused by broken or deteriorated prestressing wire. The results are recorded on a small on-board data acquisition system. The data is subsequently analyzed and used to estimate the location and quantity of wire breaks in the prestressing wire in each pipe section. The benefit of an electromagnetic inspection is that it provides a direct evaluation of that element that provides the PCCP with its strength—the prestressing wire. The wire break data can also be used to provide a ranking of pipe sections for repair or replacement based on the number of breaks in each section. When this type of data is combined with visual and sounding inspections, a good baseline assessment of a main can be obtained. A conceptual view of the electromagnetic inspection concept is depicted in Figure 2.

Figure 2. Electromagnetic Inspection of Large Diameter Mains

SOUTHWESTERN TRANSMISSION MAIN INSPECTION

Because of a history of water main failures in recent years and because of the strategic importance of the 54-inch Southwestern Transmission Main for supplying customers in the City of Baltimore and the Counties of Baltimore, Anne Arundel and Howard, the City of Baltimore requested that the initial pilot inspection program include inspection of this main. Accordingly the City scheduled a shutdown of the 54-inch main in the winter of 2005 for the purpose of assessing the main's condition. With the 54-inch main being out of service, Howard County also decided to take advantage of the scheduled 54-inch shutdown and requested that a similar condition assessment be performed on Howard County's 36-inch Southwestern Transmission Main. The reaches of 54-inch main inspected under the initial pilot inspection program are depicted in Figure 3.

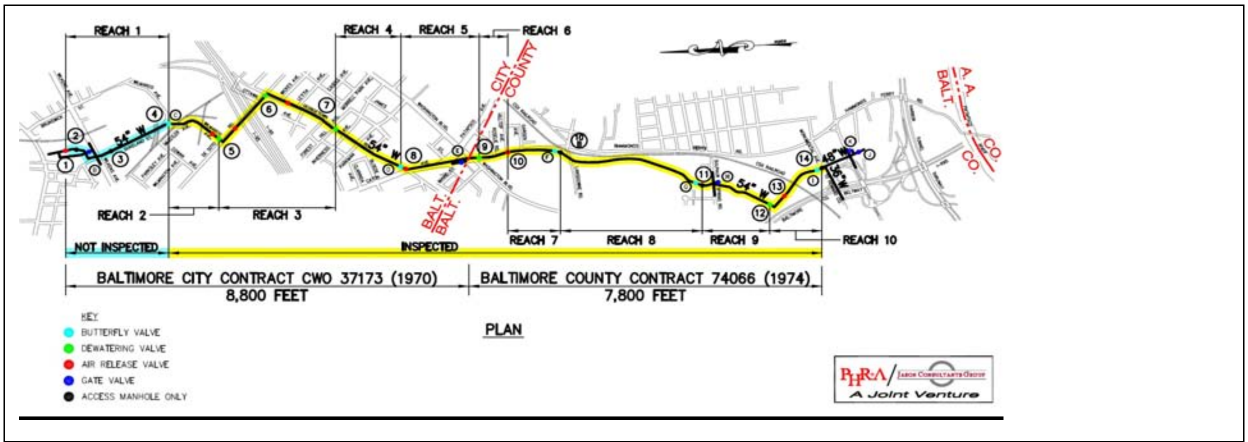

Figure 3. 54-Inch Southwest Water Transmission Main

The urgency of performing the 36-inch main inspection became even more apparent when it was learned by Howard County that the CSX Railroad planned to extend its current track system for about 3,300 feet over top of the subject 36-inch main. The main was last inspected 12 years ago. It is one of two supply mains from the Baltimore system that provide most of Howard County's water. With the 54-inch Southwest Main from Baltimore City and the 36-inch Howard County main out of service for inspection, summertime water restrictions had to be considered throughout the southwest side of the Baltimore Metropolitan Region. Figure 4 shows the reaches of the 36-inch main that were inspected.

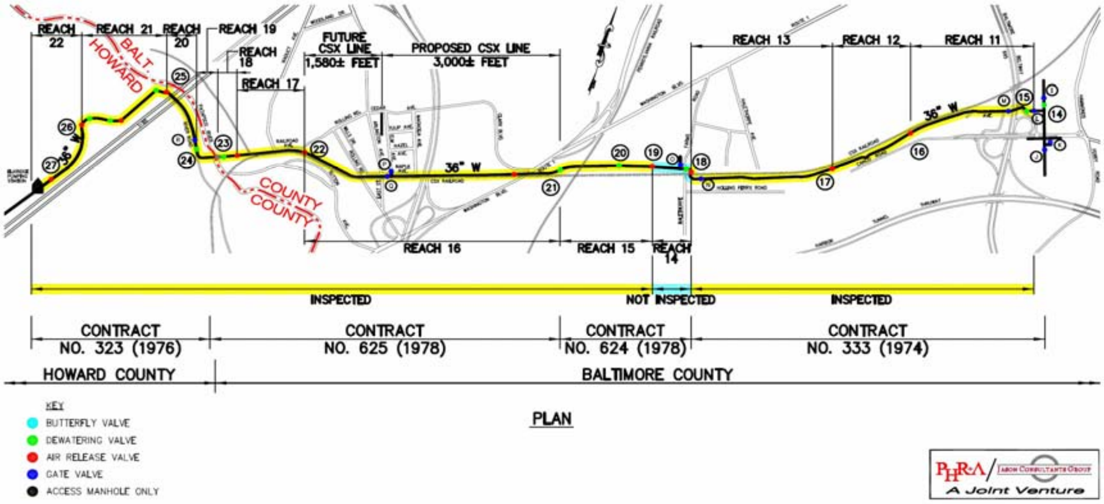

Figure 4. 36-inch Southwest Transmission Main Serving Howard County

A. 54-Inch Water Main Inspection and Results

Most of the reaches of the 54-inch main were inspected during the last week of February and the remaining reaches were completed in early April. The first of the 10 reaches originally planned for inspection could not be inspected because the field crew was unable to isolate the section for dewatering. In Reaches 2 through 5a, which are within Baltimore City itself, 456 pipes were inspected. One pipe had two longitudinal cracks, but no hollow areas were detected. Nine percent were found to have some wire breaks. Overall, the inspection results suggested that the condition of these reaches ranged from fair to satisfactory.

That was not the case in Reaches 6 through 8 in Baltimore County where a total of 253 pipes were inspected. Nineteen of these pipes were found to have significant longitudinal cracks and large hollow areas; pipes in this condition are indicative of incipient failure and could cause significant damage to nearby infrastructure or perhaps loss of life. One pipe was found with a longitudinal crack, but no hollow areas. Over forty percent of the pipes were found to exhibit wire breaks as a result of electromagnetic testing. Most, if not all, of the pipes with greater than 100 wire breaks detected by electromagnetic methods also exhibited significant longitudinal cracking with hollows.
In Reaches 9 and 10, a total of 167 pipes were inspected. Eighteen pipes were found with significant longitudinal cracks and large hollow areas, one with only longitudinal cracking and no hollow areas.

The inspection results for the 54-inch main are summarized in Table 1. Figure 5 shows a plan and profile of a portion of Reaches 7 and 8 with locations of segments where significant wire breaks were detected.

Table 1. Summary of 54- Water Transmission Main Inspection Results

REACH	PIPE NO. OF PIPES INSPECTED	NO. OF PIPES W. LONGITUDINAL CRACKS	PERCENT OF PIPES W. WIRE BREAKS
2 to 5	456	1	9
6 to 8	253	19	43
9 & 10	167	18	20

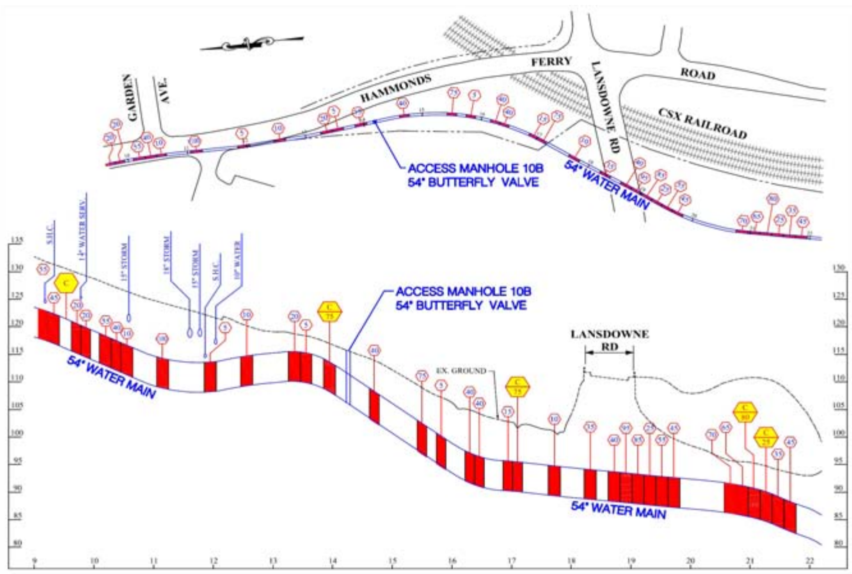

Figure 5. Results of 54-inch Main Wire Break Survey -- Reaches 7 and 8

B. 36-Inch Water Main Inspection and Results

The initial reaches of this main were inspected during the last week of January 2006, and all reaches completed by the last week of February. A total of 474 pipes were inspected in Reaches 11 through 15, from the connection at the 54-inch main to the CSX Railway crossing. Reach 14 could not be inspected because crews were unable to dewater the pipe. However, 13 pipes in the remaining reaches were found to have longitudinal cracks and four percent showed wire breaks in the surveys. Four hundred pipes were inspected in Reaches 16 and 17. These reaches paralleled the CSX rail line where five pipes were found to have longitudinal cracks, and eight percent of the pipes in these reaches were found to have wire breaks. Reaches 18 through 22 (Howard County Portion), which contained 328 pipes, was also inspected. One pipe was identified with longitudinal cracks in these reaches, and 17 percent were found to have wire breaks. The results of the inspections are summarized in Table 2.

Table 2. Summary of 36-Water Transmission Main Inspection Results

REACH	PIPE NO. OF PIPES INSPECTED	NO. OF PIPES W. LONGITUDINAL CRACKS	PERCENT OF PIPES W. WIRE BREAKS
11 to 15	474	13	4
16 & 17	400	5	8
18 to 22	328	0	17

ACTION PLAN & JURISDICTIONAL IMPLICATIONS

By early March, the majority of the visual, sounding and electromagnetic inspections for both the 54-inch and 36-inch PCCP pipes were completed. Although a complete analysis of the data collected by then would not be available until later in the summer, and calibration of the data was being deferred to a later date, a preliminary evaluation of the results of inspections showed that major sections of both the 54- and 36-inch mains were at risk of failure. A meeting of the jurisdictions was called at that time to discuss the preliminary results. After reviewing these results, Baltimore City, the three affected counties and PHR+A / Jason agreed that:

- The 54-inch and 36-inch mains that were inspected must not be returned to service in their current condition.
- The extent of pipe deterioration along Reaches 6 through 8 of the 54-inch main dictates the complete replacement of the entire main in these reaches.
- Selected 54-inch pipes in Reaches 2 through 7, and 9 and 10 also need to be replaced or repaired
- The 36-inch main beneath the CSX extension must be repaired as soon as possible

It was apparent that the situation being faced had significant implications for all four jurisdictions. The 54-inch Southwest Transmission Main is a major water supply feed to the southwest side of Baltimore City and Baltimore County, Northern Anne Arundel County and Eastern Howard County. The 36-inch transmission main supplies one-third of Howard County's summer water requirements. With an estimated one year's time to complete all pipe repairs and replacements, the meeting's attendees had to quickly develop plans to deal with the reduced water supply to customers and expedite completion of the necessary construction work by all means possible.

Major elements of the plans that were developed included the following:

- During periods of peak spring and summertime demands, localized water restrictions to several, if not all jurisdictions, would be required while the 54-inch and 36-inch mains were out of service.
- Baltimore City would install a temporary bulkhead on the 54-inch main to permit water service to be returned to a critical reach of the main once selected repairs had been completed.

- Baltimore County would immediately institute a fast-track rehabilitation and replacement program for the 54-inch main
- Howard County would implement a fast-track rehabilitation program for the CSX Reach of the 36-inch main.
- Jurisdictions would work with one another to explore possibilities of providing temporary supplemental supplies to Anne Arundel and Howard Counties.
- Close coordination between affected jurisdictions would be required throughout the period that the 54-inch and 36-inch mains were out of service.
- Periodic inter-jurisdictional meetings would be held to coordinate activities and to communicate progress.

EMERGENCY REHABILITATION/REPLACEMENT PROJECTS

Based upon the aforedescribed action plans, three major projects were identified as immediate priorities with the installation of the temporary bulkhead and carbon fiber repair of several lengths of 54-inch pipe being the most immediate.

A. Bulkhead / Buttress for and Carbon Fiber Repair of 54-inch Main

During the pilot inspection program, Baltimore City had become aware of inadequate pressures in portions of the service area that supplied water to critical facilities. The facilities in the area included the St. Agnes Hospital, one of the State of Maryland's largest hospitals. To mitigate any further pressure problems in this area while repair work on Reaches 6 through 10 proceeded, the City decided to first construct a temporary bulkhead and split buttress on the 54-inch pipe at the intersection of Washington Boulevard and Hammonds Ferry Road. In addition, the City decided to repair selected distressed pipes in the area by lining with carbon fiber. The design for this work was initiated in late April 2006 and completed in May 2006. Construction of the carbon fiber pipe repairs took place in June 2006 and construction of the bulkhead and buttress began in May and by August 2006, 7,000 linear feet of 54-inch main was returned to service. Figure 6 shows a photograph of the laminated carbon fiber lining being applied and Figure 7 depicts the location plan prepared for the bulkhead/buttress. The estimated cost for this work was $0.5 million.

Figure 6. Lining of 54-inch PCCP Main with Carbon Fiber

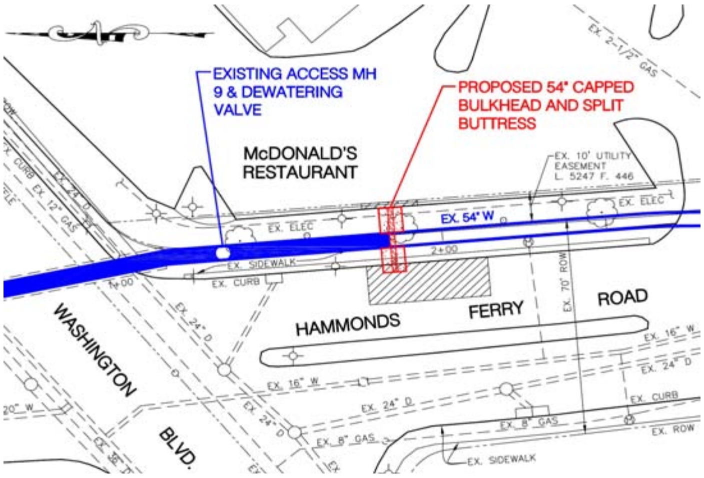

Figure 7. Plan of Bulkhead and Buttress for 54-inch Pipe

B. 54-inch Main Repair/Replacement Project

The objectives of this project, carried-out by Baltimore County, were to repair or replace approximately 5,700 linear feet of pipe and return it to service by the end of the Spring of 2007. This project was in three parts:

- Replace approximately 5,000 feet of 54-inch main.
- Slipline 330 feet of 54-inch main.
- Repair twenty 54-inch pipes with carbon fiber lining.

The design for this project was initiated in late April 2006 and contract documents were completed in September 2006. Bids were received in December 2006 and the construction contract Notice to Proceed was issued in late January 2007 with a target of returning the main to service no later than mid-June 2007. Extraordinary efforts were made with the pipe vendor to ensure the quality of the replacement PCCP and its availability in time to meet the project's schedule. Among these efforts was the placement of an inspector under contract to the County to monitor quality control at the vendor's plant in Palatka, Florida while the pipe was manufactured. Figure 8 shows prestressing wire being placed on 54-inch newly cast PCCP at the pipe manufacturing plant. The estimated construction cost for this project is approximately $11 million.

Figure 8. Pre-stressing Wire Being Placed on Newly Cast 54" PCCP

C. 36-inch Main Repair/Replacement Project

Design of this project was initiated in June 2006 by Howard County to repair twenty-one defective 36-inch sections that lie in close proximity to the existing CSX railway and fall under the alignment of a future CSX railway extension. The method of repair selected for this work was carbon fiber lining. This project proceeded very quickly. Contract documents were completed by another consultant engineering firm, Notice to Proceed was issued to the Contractor in early August, and construction was completed on September 1, 2006. The estimated cost of this project was $1.5 million.

SUMMARY

The project described in this paper was originally planned as a pilot inspection program to assess the condition of 17,000 feet of 54-inch PCCP transmission main, as a part of a comprehensive program by the City of Baltimore to develop a an approach to perform condition assessment for some 200 miles of large diameter transmission. In addition to the 54-inch inspection, the pilot program evolved into providing a condition inspection of 21,000 feet of 36-inch main. Visual, sounding and state-of–the–art electromagnetic methods were used to inspect 54-inch and 36-inch mains. These inspections were to perform the basis for baseline conditions assessments for planning future rehabilitation projects on the system as a whole. What transpired instead was the requirement for an emergency response to findings resulting from the inspections of these mains. It was determined that these major transmission mains serving 250,000 people in the region were deteriorated to a point that they could not be returned to service without several major repair and replacement projects totaling in excess of $13 million. Planning for service interruptions in four jurisdictions were made, and although some water restrictions became necessary, water service was maintained while these major projects were underway.

A Proactive Approach to Asset Management: Milton's Town-Wide Sewer Investigation & Rehabilitation Program

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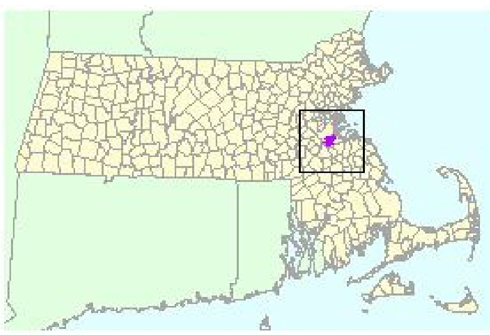
Introduction

In order to protect their substantial investment in wastewater collection system infrastructure and to meet anticipated upcoming regulatory requirements, The Town of Milton, Massachusetts (Milton) has taken a proactive approach towards operating and maintaining the wastewater collection system by embarking on a comprehensive Town-Wide Sewer Investigation & Rehabilitation Program. This program is a phased approach to identify wastewater collection system deficiencies, evaluate rehabilitation options and follow through with design and construction on a continuous set schedule. Identifying and correcting problems will reduce inflow and infiltration (I/I) and immediately enhance the value of the system.

Wastewater collection systems are constructed for the purpose of reliably transporting wastewater from each individual user to the wastewater treatment facility. A comprehensive annual investigation program ensures that the wastewater collection system properly serves this intended purpose on a perpetual basis. By following a regular program to investigate and repair the wastewater collection system, Milton will be in a better position for regulatory compliance, maintenance and management and addressing capital improvements.

Background

Milton is located approximately eight miles southeast of Boston and is roughly 13 square miles in area. The town is situated between the Neponset River and the Blue Hills. The current population is approximately 26,000 people, and the town is mostly residential. There is very little commercial or industrial development in Milton.



The municipal sewer system is comprised of approximately 468,000 linear feet (l.f.) of sewer ranging in size from 6 to 20-inches in diameter. There are over 3,000 manholes and seven wastewater pumping stations. The system is divided into 48 subareas. Milton discharges an average daily flow of approximately 4.8 million gallons per day (mgd).

Milton is part of the Massachusetts Water Resources Authority (MWRA), a public authority that provides regional water and wastewater services to communities in the metropolitan Boston area. The majority of the flow from the town is tributary to two MWRA interceptors: the Nut Island Interceptor and the Neponset Valley Interceptor, which enter Milton from the northwest and southwest, respectively. The interceptors meet within the boundaries of Milton to form a single 12-foot diameter sewer that exits town through the north to Quincy, and on to the Nut Island Headworks and the Deer Island Treatment Plant. The MWRA is responsible for the maintenance of these interceptor sewers. There are approximately 37 public and 13 private connections along these interceptors within Milton. MWRA operates 15 continuous recording wastewater flow meters along these interceptors to measure the majority of the flow from the town. Additional flow discharges directly to the Boston and Canton sewer collection systems at various locations, and eventually onto the MWRA system. This flow, approximately 24 percent of Milton's total wastewater discharge, is estimated by the MWRA. MWRA sewer charges to Milton are based partially upon the measured plus the estimated flow.

Previous Projects

Milton has completed several projects to identify and remove excessive inflow I/I. A Wastewater Facilities Plan (WFP) was completed in 1983, and revised in 1986. In the WFP, sewer subareas were delineated and I/I rates were estimated for each subarea. Town personnel completed some building inspections in the 1980s.

In 1991 an I/I Analysis and Phase I Sewer System Evaluation Survey (SSES) was performed on subareas that were identified as having excessive I/I in the WFP. The I/I Analysis and SSES included:

- Flow monitoring of approximately 227,000 l.f. of sewer
- Flow isolation of approximately 183,500 l.f. of sewer
- Approximately 350 manhole inspections
- Preliminary cost-effectiveness analysis for justification of television inspection
- Smoke testing of approximately 141,300 l.f. of sewer

A Phase II SSES was conducted in 1994 and included television inspection and a final cost-effectiveness analysis for I/I source removal. I/I rehabilitation projects were completed in 1995 and 1997, which included 63,770 l.f. of pipeline rehabilitation.

In spite of the I/I work performed, Milton still had one of the highest percentages of infiltration of the MWRA communities in the late 1990s. While substantial I/I Analysis and SSES work was conducted, much of this work was scattered throughout town and did not achieve a marked reduction in infiltration. It was time for Milton to direct resources into identifying specific I/I sources and move forward with a solution. Prior to looking at the system holistically, however, it was necessary to review the previous I/I work and solve known problems. In 1999, Weston & Sampson created an I/I Reduction Program. The project included administrative tasks such as:

- Evaluation and re-calculation of past cost-effectiveness analyses
- Development of a comprehensive sewerage system map
- Creation of an inventory of existing building inspection data

Subsequently an I/I rehabilitation project was conducted to repair known sources of I/I that were not addressed as part of the previous projects. While this repair, as well as previous pipeline rehabilitation, was somewhat scattered throughout town, it was necessary to complete this work before moving forward.

As part of the I/I Reduction Program, an estimated 326,000 l.f. of sewers were smoke tested. Dye testing and flooding was utilized to confirm direct and indirect inflow sources identified through the smoke testing. With this work completed, all of Milton's wastewater collection system had been smoke tested to identify external sources of inflow. Public sources were corrected and an inflow removal program was developed to target the remaining private inflow sources.

Shortly after that, the town combined a building inspection program with a water meter replacement project. This project identified internal private inflow connections to the sewer system such as sump pumps and floor drains. Water meter installers looked for these sources while they were installing the new meters. The private sources identified were targeted for removal under a private inflow removal program.

After the known I/I problems were corrected, and a thorough search for inflow sources had been completed through town-wide smoke testing and building inspections, Milton recognized the need for a systematic approach to identify and remove infiltration.

Before that, however some housekeeping was performed with the updating of the town-wide sewer map and Sewer Use Regulations. After these tasks were accomplished, the next step was to prioritize areas for investigation and rehabilitation.

Town-Wide Sewer Investigation & Rehabilitation Program

A Priority Evaluation was conducted to compile information on the collection system. Data from previous studies and rehabilitation projects performed in Milton as well as information from town personnel were used to rank sewer subareas according to the following prioritization schedule:

1) Emergency situations such as collapsed pipes or blockages, as identified by town personnel
2) Areas with high infiltration measured during the 1983 flow monitoring with no previous rehabilitation

3) Areas presenting chronic operation and maintenance problems
4) Areas not previously rehabilitated (ranked based on percentage of cross-country sewers)
5) Areas previously rehabilitated (ranked based on percentage of cross-country sewers)

The end result was a document which summarized previous work, described the goals and objectives of the program, outlined the ranking criteria and included a table showing each subarea, linear footage, ranking and reasoning for position in the plan. The table serves as a road map for the plan.

Pump stations and siphons were already included as part of Milton's regular maintenance program. Pump stations are inspected regularly, and seals and check valves are replaced as necessary. Siphons are regularly cleaned and maintained. Therefore these components of the system were not included in the priority evaluation.

The Town-Wide Sewer Investigation and Rehabilitation Program was originally developed as a 16-year plan in order to maintain a limited cost per year as requested by the town. However, a shorter program (ten years) was ultimately adopted because the initial phase of the project was publicly well received. After the first year of the program, the Department of Public Works successfully promoted the project to the decision-makers as a ten-year program, so yearly funding was increased.

The shorter program allows for sewer system problems to be addressed promptly and in a proactive manner. By pursuing a more aggressive program, I/I will be reduced at a faster rate and problem areas will be addressed in a timely fashion. If Milton continues to assess the entire wastewater collection system over a ten-year period, approximately 50,000 linear feet of sewers must be evaluated each year.

The goal is to initiate the following steps in selected areas of the wastewater collection system annually:

Table 1. Annual Sewer Evaluation / Rehabilitation Program

Activity	Schedule
Evaluation & Inspection	Spring
Engineering Review/Reporting	Summer
Design	Fall
Bid & Award	Winter
Construction	Following Spring

<u>Evaluation and Inspection</u> includes sewer line cleaning, television and manhole inspection, mapping updates, and incorporating the information collected into a database. The database contains general pipeline information, manhole and television

inspection observation, rehabilitation recommendations, and infiltration rates. The database is linked to Milton's Geographic Information System (GIS).

Engineering Review/Reporting includes the review of inspection data, cost-effectiveness analysis and preliminary design of rehabilitations. Defects are categorized by four parameters:

1) *Excessive*, where the cost to rehabilitate the source is less than the transportation and treatment (T&T) cost
2) *Non-Excessive* is the opposite, where the cost to rehabilitate the source is more than the T&T cost.
3) *Value-Effective* means the cost to rehabilitate the source is more than the T&T cost, but rehabilitation is recommended because of the relative value of the repair. In this case, the T&T cost is within 10% of the rehabilitation cost.
4) *Recommended* means the cost to rehabilitate the source is more than the T&T cost, but rehabilitation is still recommended for structural repairs that are a priority

T&T costs are based on MWRA's flow based charges to Milton, and relative components of the town's sewer division budget, debt service and capital costs.

As part of the reporting phase, an evaluation of the hydraulic capacity of main sewers within the project area is performed using the Chezy-Manning equation for open channel flow. This activity serves to compile information on the system's main collector sewers and may ultimately be used to identify hydraulic deficiencies or evaluate requests by developers proposing to connect to the wastewater collection system.

The Beauty of the Program

Benefits of the program include simplicity and flexibility. There is no expensive or complicated software to buy or learn. A GIS program was implemented in 2005. The GIS has enabled Milton to better track the condition of the wastewater collection system and the repairs performed. They can also use the GIS to track the status of the program itself, and to determine where each subarea falls within the scope of the program. The GIS has enhanced the program. In addition, the program is flexible so Subareas can be re-prioritized depending on the Milton's needs and available funding.

Developers looking to tie into the system are given the opportunity to contribute to the program through the flow-based I/I and Capacity Fee. Fees collected go to a dedicated sewer mitigation account (Enterprise Fund) for work and projects related to I/I removal or capacity improvements. Because of the consistency of the program, projects are always available to be undertaken by developers, and subareas affected by proposed development may be analyzed easily and adjusted within the lifecycle of the program if necessary.

Aside from cleaning and maintaining the wastewater collection system on a regular set schedule, Milton benefits from the program because there is almost always a sewer service or construction contractor in town with equipment capable of handling emergency situations as they may arise.

The program is also consistent with the Environmental Protection Agency's (EPA) proposed Sewer System Overflow (SSO) program and proposed Capacity, Management, Operation & Maintenance (CMOM) regulations. As per the proposed rule, all collection system owners would have certain responsibilities. There are six main components of the program:

1) CMOM – Assurance that the collection system can collect and transport all base and appropriate peak flows with a management program that addresses program goals; organizational description of the system; legal authority; specific design and installation provisions; and monitoring and modification of the program
2) Prohibition (affirmative defense)
3) Record keeping, reporting and public notification
4) Remote treatment facilities
5) Satellite collection systems
6) Watershed management principles

While this regulation has not yet been enacted, State and Federal regulators are using these components as benchmarks for basic sewer system management and performance. The town is well prepared to comply with these regulations when and if they are enacted.

Where Are We Now?

The Priority Evaluation document was prepared in 2000 and the program began in 2002. So far, the town has completed the investigation of Years One through Four of the program. Rehabilitation associated with Years One through Three has been completed and construction of the Year Four Rehabilitation is currently underway. Year Five investigations will begin this spring.

The following is a summary of work completed:

- Inspected 168,500 l.f. of sewers
- Inspected 1,000 manholes
- Rehabilitated 55,250 l.f. of sewers
- Rehabilitated 150 manholes
- Removed an estimated 418,500 gpd of infiltration

Summary and Conclusions

While a yearly investigation and rehabilitation program is an important component of the regulatory requirements described above, there are other benefits. A regular investigation and rehabilitation program can achieve the following:

- Protect the capital investment in the wastewater collection system components and equipment
- Prevent public health hazards and minimize damage to public and private property
- Provide efficient use of funding through reduced operating and capital costs
- Promote a safer work environment and fewer accidents and worker's compensation claims
- Reduce liability and costs for claims associated with backups and overflows
- Gain public approval and support for rehabilitation and capital improvement projects
- Benefit to the environment
- Create prompt and efficient service to customers
- Correct historic maintenance problems such as backups to private property and surcharging
- Reduce pump station wear and tear
- Establish a proactive and preventative maintenance operating mode for efficient wastewater collection system management
- Reduce I/I and utilize the full hydraulic capacity of the wastewater collection system
- Provide valuable long term information on the wastewater collection system

Proper wastewater collection system management is perhaps the most important component in a well-run system. The primary goal in wastewater collection system management is to decrease the amount of blockages, back-ups or overflows through regular inspection, cleaning and rehabilitation of sewer lines and manholes.

The secondary goal is to reduce problems that could affect residents. Proper record keeping and frequent evaluation also allow for identification of problem areas that may require more routine inspection or repair. This also results in a decrease in costs since emergency maintenance is usually more expensive than planned maintenance.

Another goal is to collect information on the wastewater collection system as the first step toward developing a comprehensive Infrastructure Management Program. Inventorying system components is a key factor in determining the condition of assets. These practices convey benefits by minimizing unanticipated failures and ensuring budget is available for replacement and repair, as well as emergencies. These features complete the circle of the Infrastructure Management strategy by bringing the program in line with the previously stated goals.

The Yearly Town-Wide Sewer Investigation & Rehabilitation Program allows Milton to operate in a proactive, rather than reactive mode. Problem areas are addressed quickly and efficiently. By inspecting portions of the sewer system on a regular basis, Milton is able to compile information on the condition of the sewer infrastructure, direct rehabilitation efforts appropriately, and develop a comprehensive database of the system over time.

Proper attention to each of these components, along with financial controls and capital planning, promotes proper operation and maintenance of the collection system and provides many benefits to the Town of Milton.

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Establishing a Collection System Baseline Condition Assessment Program One Step at a Time

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Abstract

This paper includes a case study for a baseline condition assessment conducted for the Erie County (N.Y.) Department of Environment & Planning (ECDEP), Division of Sewerage Management (DSM). This project was initiated when the County started looking for more efficient ways to develop their CIP for wastewater facilities, including wastewater treatment plants, pump stations, and collection systems. While the County had previously developed a GIS database for their wastewater collection systems that included information on some of their buried assets, the amount of information available for each of the 77 minisystems (defined as a sewershed or subdivision of a sewer district) varied greatly. For several minisystems there was a significant amount of useful information and for others, there was considerably less information. The baseline condition assessment approach was used to place all minisystems and trunk and interceptors on an equal-footing so that potential projects could be identified and prioritized, while still allowing more detailed information to be added in the future.

Introduction

For many utilities, the task of collecting and organizing buried infrastructure data for informed decision-making can be daunting. For Erie County, New York, this task is especially difficult due to various mergers and acquisitions of outlying conveyance systems over the years and several districts varying in age and condition. Many of the storm and sanitary sewers were installed long ago and record documents are either not available, not updated, or in a condition that makes them impossible to read. The condition of many buried assets is unknown as limited O&M budgets have not allowed for inspection of the entire system.

ECDEP DSM is responsible for servicing a large portion of Erie County with a network of eight sewer districts consisting of over 800 miles of storm and sanitary sewers, 87 wastewater pumping stations, three storm water pumping stations, five overflow retention facilities, and seven wastewater treatment plants. Motivated by the desire to develop a more systematic approach to capital improvement planning, DSM contracted with Malcolm Pirnie Inc./Red Oak Consulting (Consultant), with subconsultants GHD, LLC; and ELM Consulting to develop an objective process for ongoing CIP development. The preliminary step of this program was performing a BCA of DSM buried infrastructure assets, including the storm and sanitary sewer conveyance systems.

A comprehensive industry-leading collection system asset management program involves significant effort and would include the following:

Criticality Assessment. The criticality of the conveyance system asset can be determined based on established comprehensive methodologies such as WRc ratings that take into account the consequence and cost of failure. Several important considerations for establishing the criticality of a pipe include pipe category (i.e. interceptor sewers), depth, proximity to rivers/streams or environmentally sensitive areas, proximity to other critical assets (schools, hospitals, etc.) and difficulty of access (i.e. location under busy traffic routes, buildings, or railways). Criticality also considers other factors as available such as flow rates/capacity, likelihood of failure, and system redundancy.

Condition Assessment. A detailed physical condition assessment would ideally include closed-circuit television (CCTV) inspection starting with the oldest and/or most critical assets, with representative inspection of the system. Condition assessment is based on established methodologies, such as the National Association of Sewer Service Companies (NASSCO) Pipeline Assessment and Certification Program (PACP). PACP provides standardization and consistency for the evaluation of sewer pipe conditions and management of CCTV inspection results. A comprehensive database is created for ranking pipe conditions.

Together, the criticality and condition assessments would be used to identify areas of the collection system that require capital improvements and develop a prioritization of collection assets so that the most critical areas in the poorest condition or highest probability of failure receive first consideration in capital project implementation.

Given the extent of the County's facilities and the desire to develop a consistent process for ongoing capital improvement plan development, the main focus of this project was not a comprehensive collection system asset management program but rather to provide a foundation for the collection system CIP through completion of a BCA. The objective of the BCA was to perform an evaluation of the condition of the existing buried assets and to estimate the remaining useful life (based on pipe material and age) of major interceptors and trunk sewers, as well as establish an overall condition or rating for each "minisystem" (defined as a sewershed or subdivision of a sewer district). The estimated remaining useful life is then used to estimate and predict the priority of replacement or rehabilitation projects. This paper will focus on the wastewater collection system evaluation.

Methodology

The framework for completing the collection system BCA relied on consolidating DSM's institutional knowledge, as well as DSM's existing GIS information and data from recently-inspected areas in the collection system. This data came in the form of notes made by DSM's sewer district personnel on minisystem maps providing relative rankings of criticality and physical condition for a majority of the pipe reaches in the wastewater collection system, as well as the review of more detailed recent collection system inspection reports.

The evaluation and prioritization process included completing a system inventory, managing the information collected, and using interviews with DSM sewer district staff and engineering consultants familiar with other facilities to estimate the age and estimated useful life for different pipe materials. Each of these steps is detailed as follows.

System Inventory

The system inventory information was obtained from the County's GIS system and "mark-ups" of the collection system, along with other reports prepared by the DSM. The Consultant obtained the County's GIS shape files/Access database for all sewer districts. The GIS data was further supplemented with data on pipe information, such as installation date, pipe material, and other pertinent information that existed within the County Computerized Maintenance Management System (CMMS).

DSM provided two sets of hardcopy maps for all minisystems within County control. One set consisted of the District staff's criticality opinion for each buried pipe in the collection system; the other set showed their physical condition opinion for each pipe in the minisystem. This information was entered into the GIS for use in further evaluations.

Three main asset categories were selected for evaluation: main interceptors, trunk sewers, and minisystems. Preliminary information obtained for the three collection system asset categories included:

1. Main Interceptors – Main interceptors were defined as the major pipes that convey wastewater to the treatment plants and pump stations. In Erie County, the interceptors ranged in size from 12 inches to 60 inches. Available GIS data included size and length for all interceptor segments, material information for approximately 20% of the pipe reaches, and very minimal pipe age information. The overall condition of the main interceptor reaches was obtained from the DSM's hand marked-up drawings.

2. Trunk Sewers – Trunk sewers ranged in size from 8 inches to 24 inches and typically conveyed flow from the smaller diameter pipes found in residential areas to the main interceptors. Sizes of some reaches were available within the GIS; however, pipe material and age information for the trunk sewers were generally unknown.

3. Minisystems - DSM has defined a minisystem as a sewershed that contains no more than 999 manholes. Available GIS information documented the size of many reaches, but did not include all pipes. The existing ECDEP GIS database included the following information:

Manholes were not identified as a separate asset category in the initial BCA as it was assumed that they are generally of the same age and condition as their connecting pipes.

With little pipe age and material information for Interceptors, Trunks, and Minisystem pipes available electronically, additional data collection by other means was required. The Consultant researched pipe age and material information from DSM's existing record drawings. These were reviewed drawing by drawing. Following completion of the record drawing review, gaps in the data were identified and additional data was obtained from each of the sewer districts through face-to-face interviews with district staff. The additional information was input into the existing GIS.

The following ratings were used by DSM on the conveyance system hardcopy maps:

Criticality. Criticality evaluations were based on the district staff's opinion of the risk and liability associated with each conveyance system pipe segment and took into account functionality, health, safety, environment, future development, consequence of failure, and redundancy. Three levels were assigned:

- 1 – Not critical.
- 2 – Moderately critical.
- 3 – Very critical.

Condition. Condition evaluations were based on the DSM's familiarity with the collection system through daily operations and maintenance records, complaints and other condition assessment records. Three levels were assigned:

- 1 – New or excellent condition.
- 2 – Moderate deterioration.
- 3 – Virtually unserviceable/very significant deterioration.

In this case, DSM had already started the inventory process within their GIS system and had compiled additional information from each of the sewer districts that provided a basis for the CIP prioritization process. Therefore, the Consultant was able to use the existing information infrastructure to aid in the BCA. In other locations where GIS is not currently implemented and where there is limited information on the conveyance system, some initial work would need to be completed to establish a system of organizing and documenting system assets for further prioritization.

Managing the Collected Data

Once the initial data collection was completed, the Consultant developed a systematic approach to estimating the pipe condition, age, and material for those pipe reaches in which little or no information was available. While initial assumptions on pipe condition, age, and material were made for the purpose of the BCA, the process allows for further refinement in the future as the County conducts additional sanitary sewer evaluations.

Different approaches were taken for interceptors and trunk sewers as opposed to minisystems. For interceptors and trunk sewers (considered the most critical piping due to their size and functions), the age, pipe material, and condition were documented on a reach-by-reach basis. Therefore, individual reaches could be prioritized for potential future improvements.

Minisystems were evaluated from a broader perspective. The Consultant gathered the information known about each minisystem and applied several assumptions to each:

- Minisystems with relatively uniform age and pipe material were identified and the entire minisystem was assumed to have the same general pipe material and age.

- Minisystems with no single predominant pipe material and various pipe ages were subdivided into three pipe material categories. The pipe material categories were then divided into two age categories as shown in Table 1. The condition ratings (1 through 3) assigned by DSM on the information provided to the Consultant were then apportioned to the entire minisystem based on pipe age and material. The quantity of pipe with the worst condition rating (3) was assigned to the oldest pipes within the system. The total length of minisystem piping was then apportioned into each of the 7 pipe age categories.

Table 1: Assigned Pipe Age Categories[1]

Pipe Material	Years in Service	Assigned Median Age
VCP/CP	>50	65
VCP/CP	40-50	45
VCP/CP	<40	35
ACP	>25	30

[1] The following pipe materials are defined: VCP = Vitrified Clay Pipe, CP= Concrete Pipe, ACP = Asbestos Cement Pipe, PVC = Polyvinyl Chloride Pipe. Other pipe materials were present, but in such small quantities that the predominant pipe materials defined above were used in characterizing the collection system.

ACP	<25	20
PVC	>20	25
PVC	<20	15

Minisystem pipes with criticality ratings of 3 (as defined by sewer district staff) were itemized separately, as repair/replacement may be required for these sewers in the near future. Our evaluation showed that less than 50 minisystem pipe reaches were categorized as criticality 3. Because these pipes are already "broken out" from the general minisystem piping, they are in the format given for interceptors and trunks. As more detailed information is obtained, the GIS can be updated to reflect the changing conditions.

Estimated Remaining Life

The remaining useful life for pipes in each asset category were estimated using the following approach:

Estimated Remaining Life = (Estimated Service Life – Assigned Median Age)

The estimated useful lives for each pipe category were determined from multiple sources, including DSM's and their district staff's experience with each asset and engineering consultants familiar with other wastewater facilities in similar climates. The preliminary estimated useful lives for each type of pipe are summarized in Table 2. These preliminary values will be further refined as the work progresses.

Table 2: Estimated Pipe Useful Service Life

Pipe Material	Service Life (years)
VCP	50
RCP/CP	50
ACP	40
PVC	60

The number of years in service was determined for each interceptor/trunk sewer reach and each minisystem using the available pipe installation dates.

Adjustment factors were developed to differentiate pipes with the same age and material, but with different pipe conditions. The adjustment factor effectively reduces the remaining useful life by considering the condition of the pipe and other factors such as known physical deficiencies observed as the result of previous inspections, ongoing maintenance programs, capacity limitations, and other conditions which may affect the expected remaining useful life of the particular asset. Table 3 presents the adjustment factors used, based on the condition ratings. Adjustment factors are specific to DSM's collection system, based on personnel experience, and could vary for other utilities. For example, if a pipe was 20 years old and its condition rating was 1, there would be no reduction in the remaining useful life. If the pipe was 20 years old and the condition rating was 3, the adjusted remaining useful life would be cut in half. Any asset found to be in service longer than its expected service life yielded a negative remaining useful life.

Table 3: Condition Adjustment Factors

Condition Rating	Condition Factor
1	1.0
2	0.8
3	0.5

Results and Conclusions

Main interceptor and trunk sewer information was summarized by manhole to manhole segments on a county-wide basis. The summary table included a listing of all segments comprising the interceptors including available pertinent pipe data (installation date, size, pipe material), and physical condition ratings. A portion of the results is illustrated in Table 4. These results are sorted by estimated remaining useful life with the reaches having the values being assigned a higher priority for further evaluations. For example, a pipe with a remaining useful life of 3.2 years would be higher priority than a pipe with a remaining useful life of 22 years. As more data are collected, the adjustment factor range can be further expanded to reflect smaller or greater differences in pipe conditions as more and more conveyance system inspections are completed.

Condition and criticality information gained during this evaluation has been input into the County's GIS for easy retrieval. Presentation in this format allows the County to see at a glance which interceptor or trunk sewer reaches may be in poorer condition. This information will be combined with the results of a business driver evaluation (identifying other political, socioeconomic, and operational drivers affecting the DSM) to select projects for inclusion in future CIPs. However, in the meantime, the results provide a good starting point for prioritization of future projects, as well as identification of information gaps, which could prompt additional evaluations to gain the missing information.

Minisystems were summarized on a more general basis, by identifying either entire minisystems or portions of minisystems sharing common characteristics such as pipe age and/or material of construction. A portion of the results for minisystems is illustrated in Table 5. The results of the minisystem analysis indicate the approximate linear footage for each pipe condition and material and the estimated remaining useful life for each pipe material. While not complete, these tables can be used to compare minisystems with other minisystems throughout the County and not just within the same sewer district. In general, the tables generated can provide a good estimate of the approximate linear footage in each minisystem that may be approaching or has passed the end of its useful life. As such, the total lengths in each category can be used to prioritize portions of minisystems for further investigation and rehabilitation. As more and more evaluations are completed in the future, the additional information can be input into the County's GIS to provide increasingly more accurate representations of conveyance system asset conditions. Ultimately, the collection and presentation of the minisystem data will start to approach the presentation format used currently for interceptors and trunk sewers with the ultimate goal of having each pipe segment in the County individually itemized and tracked for condition.

Overall, the conveyance system BCA attempted to:

- Capture and organize the County's existing information and institutional knowledge in one spot.
- Define the estimated remaining useful life of the assets, which is used to prioritize pipe reaches (in the case of interceptors and trunk sewers) and minisystems for further evaluation and/or rehabilitation.
- Identify the location existing critical reaches for immediate prioritization.
- Provide a baseline process by which buried infrastructure assets can be assessed and prioritized. While the Erie County conveyance system BCA is rudimentary in its first year, the potential exists for further definition of pipe conditions in future years now that the assessment framework is in place.

Next Steps

The information collected during the conveyance system BCA was used in conjunction with a parallel BCA for assets at the County's wastewater treatment plants and pumping stations. Assets with the lowest estimated remaining useful life would receive higher priority for further evaluation, rehabilitation, and replacement. Collectively, this information will be used to develop the first-iteration CIP document under subsequent tasks of the project. The conveyance system BCA is a starting point for future conveyance system assessments and is only intended to provide an initial basis for the first iteration CIP. The County is making significant progress in updating system information through ongoing sanitary sewer evaluation and rehabilitation projects. As information continues to be gathered, the overall collection system GIS will be updated with more specific and detailed information regarding pipe materials, ages, and conditions to be used in subsequent iterations of the CIP. As projects are completed and information updated, it is anticipated that the County would subsequently use a methodology similar to that described in this document in future CIP planning cycles. This will allow policies and procedures to be developed for later activities and help ensure that more appropriate detailed collection system condition information can be collected in the future for input into the CIP process.

Table 4: Example Summary of Interceptors and Trunk Sewers – All Districts

PIPE	Upstream Manhole	Downstream Manhole	District	Minisystem	Type	Pipe Length	Pipe Size	Pipe Material	Assigned Age	Est. Total Useful Life	Est. Remaining Useful Life	Condition	Adj. Factor	**Revised Est. Remaining Useful Life**
BB70D	BA903	BA8EE	1	3	Trunk	491	24	VTP	1961	50	5	3	0.5	3
BB710	BA8FF	BA903	1	3	Trunk	414	24	VTP	1961	50	5	3	0.5	3
E7933	137	136	2	3	Trunk	219	10	VTP	1962	50	6	3	0.5	3
E7934	136	135	2	3	Trunk	192	10	VTP	1962	50	6	3	0.5	3
E7935	135	134	2	3	Trunk	180	10	VTP	1962	50	6	3	0.5	3
E7936	134	133	2	3	Trunk	437	10	VTP	1962	50	6	3	0.5	3
E7937	133	132	2	3	Trunk	41	10	VTP	1962	50	6	3	0.5	3
E794D	103	102	2	3	Trunk	227	15	VTP	1962	50	6	3	0.5	3
E794E	104	103	2	3	Trunk	160	15	VTP	1962	50	6	3	0.5	3
E794F	105	104	2	3	Trunk	91	15	VTP	1962	50	6	3	0.5	3
E7950	106	105	2	3	Trunk	165	15	VTP	1962	50	6	3	0.5	3
E7955	107	106	2	3	Trunk	77	15	VTP	1962	50	6	3	0.5	3
E7956	108	107	2	3	Trunk	128	15	VTP	1962	50	6	3	0.5	3
E7958	109	108	2	3	Trunk	412	12	VTP	1962	50	6	3	0.5	3
190F17	132	130	2	3	Trunk	194	10	VTP	1962	50	6	3	0.5	3
CF856	CD6CA	CD6C9	3	2	Interceptor	157	10	RCP	1980	50	24	3	0.5	12
CF857	CD6CB	CD6CA	3	2	Interceptor	166	10	RCP	1980	50	24	3	0.5	12
CF869	CD6CC	CD6CB	3	2	Interceptor	405	10	RCP	1980	50	24	3	0.5	12
CF873	CD6CE	CD6CF	3	2	Interceptor	300	8	RCP	1980	50	24	3	0.5	12
CFA4D	CD519	CD525	3	18	Interceptor	430	36	RCP	1980	50	24	3	0.5	12
CFA4E	CD518	CD519	3	18	Interceptor	429	36	RCP	1980	50	24	3	0.5	12
CFA4F	CD517	CD518	3	18	Interceptor	423	36	RCP	1980	50	24	3	0.5	12
CFA50	CD516	CD517	3	18	Interceptor	31	36	RCP	1980	50	24	3	0.5	12
CFA51	CD515	CD516	3	18	Interceptor	494	36	RCP	1980	50	24	3	0.5	12
CFA52	CD511	CD515	3	18	Interceptor	211	36	RCP	1980	50	24	3	0.5	12
CFA8C	CD393	CD388	3	21	Interceptor	330	24	RCP	1980	50	24	3	0.5	12

Table 5: Example Summary of District 3 Minisystem Pipes

Sewer District	Mini-system	Pipe Material	COUNTY Physical Condition Rating	Linear Footage	Assigned Median Age	Est. Useful Life	Est. Remaining Useful Life	Condition Factor	Revised Est. Remaining Useful Life
3	1	VCP/CP	3	**23,341**	45	50	5	0.5	3
		VCP/CP	2	**7,432**	45	50	5	0.8	4
		VCP/CP	1	**11,723**	45	50	5	1.0	5
		ACP	1	**2,833**	30	40	10	1.0	10
		PVC	1	**11,332**	15	60	45	1.0	45
3	2	VCP/CP	3	**4,277**	35	50	15	0.5	8
		VCP/CP	2	**3,883**	35	50	15	0.8	12
		VCP/CP	1	**1,079**	35	50	15	1.0	15
3	3	VCP/CP	3	**13,795**	45	50	5	0.5	3
		VCP/CP	3	**1,414**	35	50	15	0.5	8
		VCP/CP	2	**1,909**	35	50	15	0.8	12
		VCP/CP	1	**15,069**	35	50	15	1.0	15
		ACP	1	**2,299**	30	40	10	1.0	10
		PVC	1	**11,496**	15	60	45	1.0	45
3	5	PVC	3	**69**	25	60	35	0.5	18
		PVC	1	**1,108**	25	60	35	1.0	35
3	6	VCP/CP	3	**74**	35	50	15	0.5	8
		VCP/CP	2	**340**	35	50	15	0.8	12
		VCP/CP	1	**2,464**	35	50	15	1.0	15
		PVC	1	**720**	15	60	45	1.0	45

Setting Pipeline Rehabilitation Priorities to Achieve "Best" Results – A Case Study Using Condition and Criticality Criteria

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Introduction

In these days of fiscal frugality, utilities struggle to get financial resources for pipeline rehabilitation. Once resources are dedicated, it is critical that they achieve the "best" results possible based on multiple objectives and criteria. These criteria reflect the often competing interests of different utility departments (e.g., planning, engineering, operations and maintenance), and they must consider the expectations of the utilities' customers and outside stakeholders, including the business community, regulatory officials, and environmental groups. Add politics and the media to this scenario, and the decision making process becomes complex.

This paper presents a method of developing, applying, and defending a decision making process that is technically sound, practical to apply, and reflects the values and objectives of stakeholders both within and outside of the utility. The method sets rehabilitation priorities based on available information related to condition (probability of failure) and criticality (consequences of failure) criteria that are developed and weighted through a stakeholder's involvement process. The results provide a framework for setting priorities for immediate rehabilitation action items as well as priorities for collecting additional information in areas where it is needed to support better decisions.

In coastal Wilmington, North Carolina, the need for sewer rehabilitation became a focus of the media and politically charged after several force main failures and system stoppages resulted in a series of sanitary sewer overflows (SSOs), one of which impacted beach areas on a holiday weekend. Quickly responding, the city initiated a priority-setting process using the methodology described herein that affirmed the importance of several sewer rehabilitation projects already ongoing and identified a number of high priority rehabilitation and condition assessment efforts to be started immediately. Several of the condition assessment efforts have been completed using techniques including zoom camera inspections in priority gravity sewers and ultrasonic thickness testing of several ductile iron pipe and prestressed concrete cylinder pipe force mains. These results demonstrate the successful application of this method in a politically-charged and high profile environment. The details of this case study are described in more detail later in this paper.

Priority-Setting Methodology

There are many potential objectives of a sewer rehabilitation program including restoring structural integrity, reducing infiltration and inflow (I/I), and reducing maintenance costs. Identifying which of these objectives is the highest priority or has a higher potential for beneficial results can be difficult and sometimes controversial. Setting geographic priorities for focusing rehabilitation funding can also be difficult and can include political implications.

The purpose of the prioritization process is to identify where to focus resources to inspect, maintain, and rehabilitate different areas of the system so that the most beneficial results can be achieved. Immediate investigation of every pipe and pump station is not cost-effective for many utilities that have portions of their system performing well. A more appropriate use of finite resources is to focus immediate rehabilitation on higher priority areas of the system or critical pipelines and to monitor other areas that are critical but in good condition. In addition to this short-term plan, it is important to create a long-term rehabilitation strategy that can be updated regularly and that results in phased rehabilitation of all system components. The goal of the long-term rehabilitation strategy is to proactively identify potential problem areas and fix the problems before they result in a system failure that would cause significant impacts, as consistent with typical Capacity, Management, Operations and Maintenance (CMOM) best practices.

One way of identifying pipes and other facilities that should receive the most immediate inspection or rehabilitation is to rank them in terms of their criticality (or consequence of failure) and condition (probability of failure). Assets whose failure creates a large impact on the community and environment and whose condition is the poorest will receive immediate inspection and/or rehabilitation. Pipes, manhole structures, and facilities that receive a lower criticality and condition rating will receive some level of continued monitoring but no immediate action or rehabilitation.

The prioritization process consists of five steps illustrated in Figure 1.

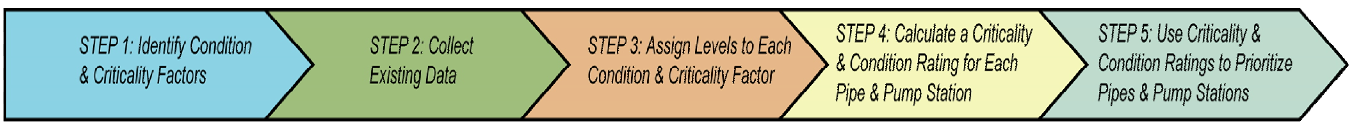

Figure 1: Prioritization Process

- **Step 1** is to identify the condition and criticality (CC) factors that will be used to assess the system. The CC factors are unique to each collection system and the weighting of these factors reflect the unique values and circumstances of the community. For example, some utilities might identify sensitive water bodies that could potentially be affected by a sewer system failure as most critical while other municipalities might identify downtown business impacts as being the most critical.

- **Step 2** is to collect the data that will be used to evaluate each factor. For the initial system assessment, existing data is typically used in the evaluation. Where additional data is needed, surrogate data (e.g., pipe age, material, maintenance history) is used in the interim and the ranking system is used as a means of identifying the highest priorities for further data collection.

- **Step 3** is to assign different levels to each factor. For example, levels of 1 through 5 may be assigned as the condition rating for a pipeline, with 5 representing a pipe that has failed or is in the process of failing. The purpose of assigning levels is so that pipes and facilities can be differentiated in terms of their condition or criticality.

- **Step 4** is to assign a CC rating for each pipe and facility. The ratings are assigned by using the level assigned to each factor and the relative importance of each factor.

- **Step 5** is to use the ratings to prioritize the system and determine short-term and long-term rehabilitation projects.

As mentioned, the factors used to develop the condition and criticality ratings should be tailored to each individual utility. There will often be similarities since most utilities have similar objectives and charges, but there will be variations based on location, conditions, and the availability of information to set the factors. Ideally, these factors are identified and weighted as part of a facilitated stakeholder involvement process. When this approach is used, the values and objectives of all interested parties are considered including different utility departments, customers, regulatory officials, and environmental and public health interests. An example of the types of factors considered to develop the condition and criticality ratings are provided in Figure 2.

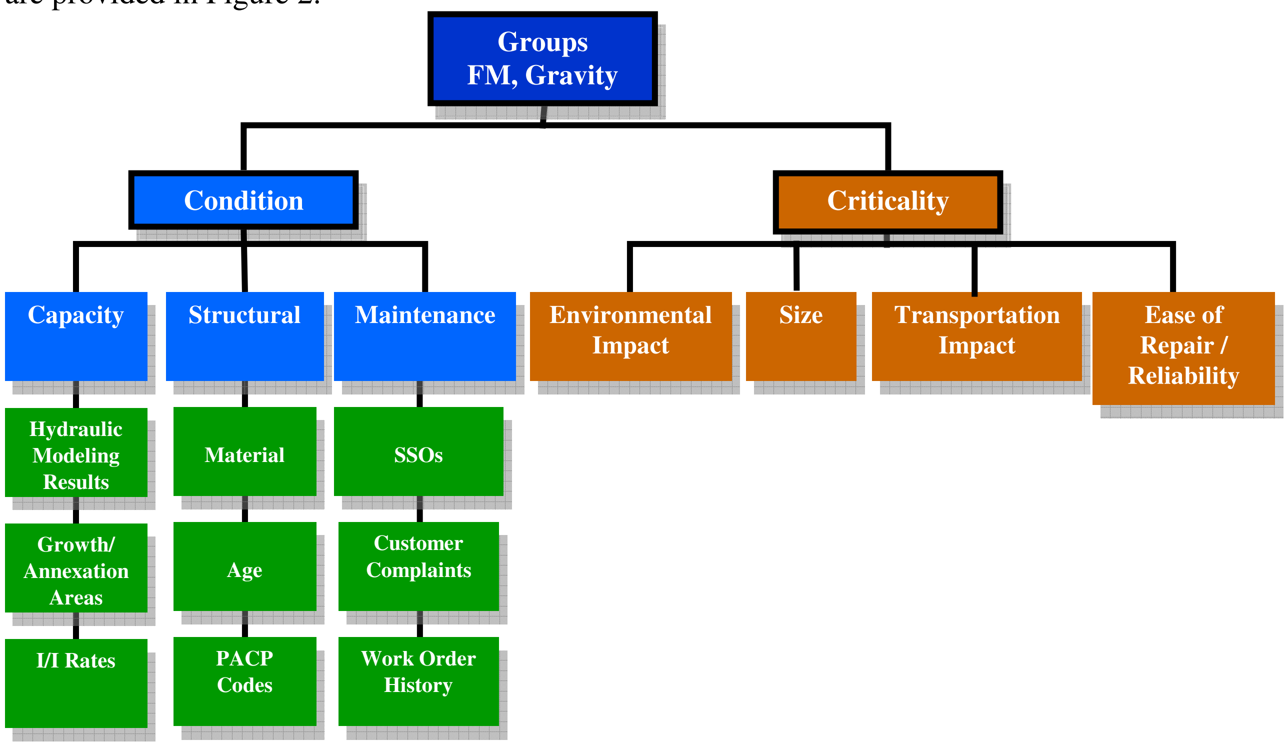

Figure 2: Examples of Condition and Criticality Factors for Setting Sewer Rehabilitation Priorities

Once the factors to be used in setting the condition and criticality are established, then "levels" must be established to differentiate between pipes or facilities related to that factor. Typically, a higher "level" is assigned to a facility that is more critical or that is in poorer condition. As an example, Table 1 provides the levels associated with the criticality factor pipe size. The level assigned increases as the consequence of failure or probability of failure increases. This is based on the reasoning that a failure in a 24-inch gravity interceptor can result in more wastewater being released than a failure in an 8-inch pipe. Therefore, the larger diameter pipe has a higher criticality based on the amount of flow conveyed. In this example, the 24-inch interceptor would be assigned a high level (5) for the quantity of flow conveyed criticality factor, and the 8-inch gravity pipe would be assigned a low level (1) for the same factor. In this example, a level of 3.1 is used for unknown diameter so that this factor does not skew final weighted ratings to the high or low side until the data is collected. The 0.1 is added to distinguish this rating from a rating of 3 so it is clear that this data is unknown.

Table 1. Example of Using Pipe Size to Establish Different Levels of Criticality for Gravity Sewers

Pipe Size (Diameter)	Level	% of Total Pipe Length
< 8-inch	1	5
8 to 10-inch	3	84
≥ 12-inch	5	10
Diameter Unknown	3.1	1

Levels must also be assigned for each condition related factor. In systems where actual condition assessment information, such as pipeline assessment and certification program (PACP) defect codes (NASSCO, 2001) are not available for the entire system, surrogate information may be used in place of actual condition information until a system assessment can be completed. Setting priorities for completing the system assessment can be another objective of this process.

Pipe material is often used as a surrogate factor for condition, and levels can be set based on historical system information related to which pipe materials have resulted in more failures. In some cases, factors can be combined, such as a ductile iron pipe in a corrosive soil environment, to identify condition assessment priorities. Table 2 provides an example of the levels assigned to different pipe materials by the City of Wilmington, North Carolina, based on the utility's failure history with these materials. Most failures had occurred as a result of corrosion problems in cast iron, ductile iron, or reinforced concrete pipe or other structural failures in vitrified clay pipe. Therefore, they assigned a higher level (resulting in a higher condition ratings or higher priority) for these materials. This initial surrogate condition information can be used as a means of setting priorities for the collection of actual condition data. Then once the actual condition information is available, in the form of defect codes for example, the actual data would replace the surrogate condition data.

Table 2. Pipe Material Levels for Gravity Sewer and Force Mains

Structural Condition Category				
Pipe Material	**Description**	**Level**	**Gravity Sewer % of Total Pipe Length**	**FM % of Total Pipe Length**
HDPE, PVC	High density polyethylene / Polyvinyl chloride	1	21	22
PCCP, TRUSS	Pre-cast concrete	3	3	12
CIP, DIP, RCP, VCP	Cast iron, Ductile iron, Reinforced concrete, Vitrified clay	5	70	66
Unknown	Material unknown	3.1	6	0

After a level of 1 to 5 is assigned to each pipe or facility (e.g., pump station) for each of the CC factors, an overall criticality rating and an overall condition rating are calculated for each system component. The ratings are also based on a scale of 1 to 5, with highest ratings assigned to those components that have the highest consequence or highest probability of failure.

The criticality rating is calculated using the levels assigned to each criticality factor (e.g., pipe size, transportation impact, environmental impact, public health impact, and ease of emergency repair) and their relative importance. Similarly, the condition rating is calculated using the levels assigned to each condition factor (structural condition, maintenance frequency, and capacity) and their relative importance. In addition, some condition and criticality factors were designated as ‘override factors’. For these override factors, if the level assigned is 5, then the condition or criticality rating will be 5, regardless of the levels assigned to the other factors.

The relative importance is the weighting, expressed as a percentage, applied to each factor in order to calculate an overall rating. The initial relative importance values are based on input received in the stakeholder involvement process. These values are then refined during the calibration with actual system data. The calibration is performed so that criticality and condition ratings are distributed across the full range of values (1 to 5) to the extent possible. This way, there is a clear understanding of the relative probability and consequence of failure for each system component. The calibration process also ensures that areas of the system with poor condition and high criticality are properly identified as being a priority.

The combination of condition and criticality ratings determines priorities for repair or replacement of system assets. This will provide the utility with a plan for focusing the available resources and funding on the most immediate needs. Figure 3 is an example matrix showing the recommended course of action for each sewer system component based on the combination of condition and criticality ratings. The actions identified by this type of matrix should be tailored to individual utility situations, and so the recommendations contained in each box of the matrix will vary by utility.

		Criticality				
		1	2	3	4	5
Condition	5	Mid Priority Program Rehab	High Priority Program Rehab	High Priority Program Rehab	Immediate Action	Immediate Action
	4	Mid Priority Program Rehab	Mid Priority Program Rehab	High Priority Program Rehab	Immediate Action	Immediate Action
	3	Low Priority	Low Priority	Regular Monitoring	Frequent Assessment	Frequent Assessment
	2	Low Priority	Low Priority	Regular Monitoring	Frequent Assessment	Frequent Assessment
	1	Low Priority	Low Priority	Regular Monitoring	Regular Monitoring	Regular Monitoring

Figure 3. Example Matrix of Recommended Courses of Action Based on Condition and Criticality Ratings

Each of the recommended courses of action are briefly described below. The specific investigative techniques or rehabilitation will vary based on the type of asset: gravity sewer, force main, or pump station.

Immediate Action

Pipes or facilities that are both very critical (criticality rating = 4 to 5) and in poor condition (condition rating = 4 to 5) are placed as the highest priority for immediate action. These assets are both more likely to fail and have high consequences if a failure were to occur. The action may include additional data collection to determine actual condition, or if enough information is known, the action would be rehabilitation.

Program Rehabilitation

Pipes or facilities that are suspected to be in poor condition (condition rating = 4 or 5), but are not as critical (criticality rating =1 to 3) should be part of an on-going rehabilitation program. These assets could be prioritized within the rehabilitation program as 'High Priority Program Rehab' or 'Mid Priority Program Rehab'. Those under the high priority rehab program are those that have a higher consequence of failure than those in the mid priority rehab program.

Frequent Assessment

Pipes or facilities that are in fair condition (condition rating = 2 or 3), but are still very critical (criticality rating = 4 to 5) should have their condition assessed frequently (every 2 or 3 years for example) since the consequences of a failure are high. The purpose of frequent assessment is to check if the condition has deteriorated to a point that the asset would be moved to the immediate action category. In many cases, assets in the frequent assessment category had a condition rating of about 3 due to unknown information about the condition. Once condition is known, and if it is determined to be poor condition, then these assets would move up to the immediate action category.

Regular Monitoring

The assets in the regular monitoring category cover a span of condition and criticality ratings that fall between the frequent assessment and low priority categories. Assets in this group cover the conditions ratings 1, 2, and 3 as in the low priority group. However, they are more critical than the low priority category since they received a criticality rating of at least 3. Because of their higher criticality, they require regular monitoring, which may correspond to assessment every 10 years for example. Some of the assets in this category are still very critical (rating 4 to 5), but are generally in better condition than the frequent assessment category because their condition rating is a 1 as opposed to 2 or 3. The activities performed under regular monitoring are the same as those performed under frequent assessment. However, the activities are not performed as often.

Low Priority

The low priority category includes assets that are believed to be in good to fair condition (condition rating = 1 to 3) and that are not considered critical (criticality rating = 1 or 2). The assets in this category will receive some level of condition monitoring to see if they need to be included in the program rehabilitation group.

Example Application

This prioritization methodology was applied in Wilmington, North Carolina, as a means of identifying immediate needs as part of a systemwide sewer condition assessment. The immediate action plan provided in Table 4 was one of the outcomes of this process. This action plan was then expanded into a detailed investigation plan.

The investigation phase of the sanitary sewer system assessment project in Wilmington focused on collecting actual condition information from the priority projects identified. This information was used to refine and enhance the Immediate Action Plan projects and develop a comprehensive rehabilitation and improvements strategy. In addition to projected costs for rehabilitation of the priority projects, this comprehensive rehabilitation strategy will further discuss the assets categorized for programmatic rehabilitation and those where additional investigation is recommended.

Field condition assessment work is ongoing. Investigative techniques of the gravity sewers include the use of zoom camera technology as a cost-effective means of performing manhole inspections and acquiring sewer condition information from approximately 50 to 75 feet in each direction from the manhole.

Table 4: Example Immediate Action Plan

Project	Description	Recommended Action
B – River Road Force Main	Approximately 18,500 feet of 20-inch and 24-inch force main, mostly PCCP.	Additional investigation of both PCCP and older DIP repair pipe to determine condition.
C – PS12 Force Main	Approximately 5,300 feet of 20-inch DIP force main.	Immediate investigation is recommended to confirm suspected poor condition.
D – PS10 Force Main	Approximately 5,300 feet of 30-inch force main, mostly DIP, elevated over Smith Creek crossing.	Immediate investigation is recommended to determine condition.
G – Burnt Mill Creek Outfall	3,100 feet of 48-inch RCP/DIP; 18 manholes.	Further investigation is recommended via manhole inspection and zoom camera technology to determine condition of the portion of the outfall downstream and including the junction box at McCumber's Ditch.
K – Downtown sewershed 11	Approximately 1,300 feet of 6-inch, 25,600 feet of 8-inch, 500 feet of 10-inch, 1,400 feet of 12-inch sewer; 131 manholes.	Further investigation is recommended via manhole inspection and zoom camera technology.

Force main investigations have included non-destructive testing and the removal of pipe coupons as a means of calibrating and verifying non-destructive testing results at critical locations. For sections of prestressed concrete cylinder pipe (PCCP), inspectors have performed visual inspections and hammer soundings at key excavated locations along the pipe length. Soil and groundwater samples have been taken to determine the aggressiveness (which may indicate potential for external corrosion). A small (approximately 12-inch by 12-inch) sample of mortar coating was chipped away and the wires exposed. The wires were visually inspected and spacing measured and the hole patched. Non-destructive ultrasonic thickness testing was performed to look for thinning due to hydrogen sulfide corrosion.

Locations along force mains with the greatest potential for corrosion were selected based on an evaluation of the pipeline profile and operational procedures. Based on the pipe profile, the sections were selected that do not remain submerged in a static condition (after the pumps shut off) since the free discharge is lower than the downhill run of pipe. In other words, in the worst case static condition (assuming potential failure of the air/vacuum release valves), these sections of pipe would drain by gravity to their respective downstream water level whether it be the discharge of the force main or the next downstream high point. This trapping of air leads to three major concerns 1) additional head losses; 2) surge pressures when the pumps restart; and 3) corrosion caused by the release of hydrogen sulfide.

In addition to this field testing, surge analysis and a thrust restraint analysis have been conducted. All of this information has been used to perform a structural analysis of the pipe and make recommendations on continued use or remedial measures that may be needed to alleviate the potential impacts of surge.

It is always a concern excavating a pipe under pressure and this must be performed with great care. In addition to safety measures being taken by the excavation subcontractor and the testing subcontractor, an emergency procedure was developed and approved prior to beginning the field assessment work.

The final rehabilitation plan and improvements strategy addresses immediate sewer rehabilitation needs, including priorities, schedules, recommended rehabilitation techniques and approaches, and estimated costs to perform the recommended work. Recommendations include a framework with which the utility's wastewater collection system rehabilitation and replacement program can be managed and implemented. This framework includes:

- Methods and technologies for collecting and coding condition assessment information needed for the purpose of assigning condition level to all wastewater collection system assets. The need to collect information in priority areas based on the consequences of system failure was considered. This included providing recommended schedules and procedures for updating condition information as necessary.

- Alternatives for delivery of the rehabilitation and replacement work and guidance on the potential use of multiple delivery mechanisms based on program objectives including traditional bids, design/build, retainer contracts, construction management at-risk and possible variations as appropriate.

- Methods and technologies for conducting rehabilitation and replacement of various utility infrastructure components.

- Description of methods for managing and the roles of in-house and external resources needed to implement the program including engineers, field assessment firms, construction firms, and other potential special program needs.

- Methods for documenting the results of the rehabilitation and replacement program in meeting the program objectives including restoring and maintaining system capacity, repairing damage and restoring structural integrity, and reducing system maintenance costs in rehabilitated areas.

References

Dalziel, Andrew and Chris Macey (2004), "The Benefits of using Quality Sewer Condition Data in the Asset Management Process," *Trenchless Technology*, Volume 13, Number 2, Peninsula, OH.

Garvin, Michael J. (2003), "Strategic Characterization of Water System Infrastructure and Management," *J. of Management in Engineering*, Vol. 19, No. 4, ASCE, New York, NY.

Miles, S. Wayne (2006), One-On-One Interview," *Trenchless Technology*, Volume 15, Number 6, Peninsula, OH.

National Association of Sewer Service Companies (NASSCO) (2001), "Pipeline Assessment and Certification Program (PACP)," Baltimore, MD.

WEF (1999), WEF Manual of Practice No. 7: Wastewater Collection Systems Management. Water Environment Federation, Alexandria, VA.

Condition Assessment Priorities for City of Houston

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Over the last few years, The City of Houston has experienced failures of various large diameter water lines. Some of the most notable have involved 36-, 42-, 48-, and 60-inch Prestressed Concrete Pipe (PCCP), and 24-, 30- and 36-inch Cast Iron Pipe.

Because of these failures, the City is concerned with the long-term performance and structural integrity of other large diameter water lines. The City requested Lockwood, Andrews & Newnam, Inc., (LAN) to prioritize the water infrastructure for evaluation under a Condition Assessment program. LAN's task is to prioritize existing lines based on the likelihood that they may have distress and the risk if a failure or water outage were to occur.

The four pipe lines above were all constructed of prestressed concrete cylinder pipe (PCCP) between 1960 and 1996, however, each failed for distinctly different reasons. The causes of failure of these lines ranged from problems associated with hydrogen embrittlement of prestressing wire, internal corrosion to structural damage caused by transient surges, and failure of lead caulking of cast iron pipe joints.

A list of criteria is organized into a GIS database in order to determine the theoretical condition of each pipe line. Physical assessment of the lines is to be scheduled based on criteria including:

- Age
- Pipe Material
- Critical nature of line (Facilities served; schools, hospitals, industry, etc.)
- Cathodic protection system
- Leak history
- Operational history
- Review of construction inspection records
- Adjacent construction projects

This paper describes the criteria used by the City of Houston to prioritize their Condition Assessment program and some of the issues and constraints faced to fully inspect existing pipe lines.

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I. GENERAL

A. Background

Over the last several years, The City has experienced failures of a water 60-inch water transmission line, near the intersection of Clay and Cullen, a 42-inch water transmission line near Sims Bayou Pump Station, and a 48-inch transmission line along Allen Genoa near Queens Blvd. These lines were all constructed of Prestressed Concrete Cylinder Pipe (PCCP), however, each failed for distinctly different reasons. In addition to these lines, an aging large diameter cast iron water line has been continuously developing leaks for many years.

Because of this, the City is concerned about similar water lines' long-term performance and structural integrity, and established a contract with LAN to prioritize and investigate lines suspect to pose maintenance problems.

Failure 1: 60-inch PCCP

At approximately 5:00 PM on Sunday, October 13, 2002, a 60-inch PCCP water line supplying the City of Houston's (City's) Central Business District, and areas to the west, suffered a catastrophic failure. In addition to the damage to the water line, this failure also caused damage to several private residences, automobiles and some commercial properties.

A manned entry inspection was not possible until the night of October 30, 2002 due to the inability to locate working isolation valves and interconnections in order to fully shut down the water line. Initial observations indicated that the problems with this line were not isolated to the one failed pipe section and a more in-depth study and rehabilitation were needed.

Based on record research and verification in the field, the 60-inch pipe material is an embedded cylinder Prestressed Concrete Cylinder Pipe (PCCP) water transmission line, constructed in the mid 1970s. The record information for the original construction project indicates this pipe material should have been manufactured in accordance with AWWA standard C301 in effect at the time of manufacture.

One of the first problems encountered involved the existing access manways constructed in the pipeline. The manholes over the access manways were only four feet in diameter and many of the manhole structure bases were poured directly on top of the manway flanges. Therefore, to gain access into the pipe for dewatering and inspection, the entire manhole structures had to be removed and rebuilt.

Second, the original installation did not adhere to the pipe lay schedule.[3] The installed the pipeline had vertical and horizontal deflections and offsets not shown on the record ("as-built") drawings and the specifically designed pipe sections were not installed in the sequence identified.

With the access manways in many cases over 1,000 feet apart, and the above mentioned issues, dewatering and visual inspection of the 60-inch pipe was a difficult and slow to complete process. Other issues that complicated completion of the initial rehabilitations and assessment included; encroachments into the City's easements, constant leaking of the isolation valves (did not totally shut the water flow into the work

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3 A lay schedule is provided on large diameter pipelines to provide the contractor with a specific sequence of installing the pipe sections in order to match the design engineer's drawings. A lay schedule will also identify the location of horizontal and vertical deflections in the joining of the pipe sections.

area), availability of pipe material[4], and the need to preserve the removed pipe intact for future testing.

Several of the excavated pipe spools have shown that most or all of the prestressing wires have failed by brittle fracture. As part of the investigation, wire samples were removed from the damaged pipe pieces and tested for tensile strength, torsion, and susceptibility to hydrogen embrittlement (HE).

The prestressing wire used in the manufacture of this 60-inch pipe shows definite signs of strain aging and hydrogen embrittlement sensitivity. Testing and analyses indicate these conditions were probably enabled by the manufacturing process, and then compounded by the effects of the magnesium cathodic protection system operation.

It should be noted that based on this investigation, all wires tested appear to have met the requirements of the time of manufacture. Several tested wire samples failed prematurely as a result of longitudinal cracks. Those cracks were most likely not present at the time of manufacture, but may have been caused by hydrogen embrittlement after installation of the pipe sections. This is a reasonable assumption because the wire would have failed under the initial wrapping stress if cracks were present during manufacture. The standards in effect when the wire was manufactured did not require testing that would have identified strain aging in the wire.

Another major issue exists regarding the fact that the pipe was not laid in sequence according to the design lay schedule. Wire pitch, which is the spacing between individual wire wraps, was measured during test excavations. This pipe was manufactured with four distinct pipe design classes, which represent different pressure ratings and overburden loading, using the same wire but changing the cylinder gage and wire pitch. Several of the pipe spools excavated did not match the design class stated in the lay schedule. In the particular cases examined, the actual pipe design was greater than the design required for the installed depth. However, it is possible that pipe designed for a shallow depth may have been installed in a deep cover situation. This error could result in excessive egging of the pipe and eventual crack development in the mortar leading to corrosion.

Failure 2: 42-inch PCCP near Sims Bayou Pump Station

During the early morning hours of June 8, 2004, an existing 42-inch water main in southwest Houston suffered a catastrophic failure. This failure occurred in a pipe section of prestressed concrete cylinder pipe (PCCP) installed in 1991 through 1992, which supplies potable surface water to an area of approximately 75 square miles. Several residential properties were flooded as a result of the failure. The City of Houston (City) authorized Lockwood, Andrews & Newnam, Inc. (LAN), to assist in the failure assessment and to design the repair for the water line.

This particular PCCP is the lined cylinder variety, composed of a concrete core, prestressing wires wrapped around a thin steel cylinder, and mortar coating. A picture of the failed pipe section is included as Exhibit 1.

4 Because of the diameter of the pipe, rehabilitation and replacement sections had to be manufactured to order. The delivery time experienced was between 6 and 8 weeks.

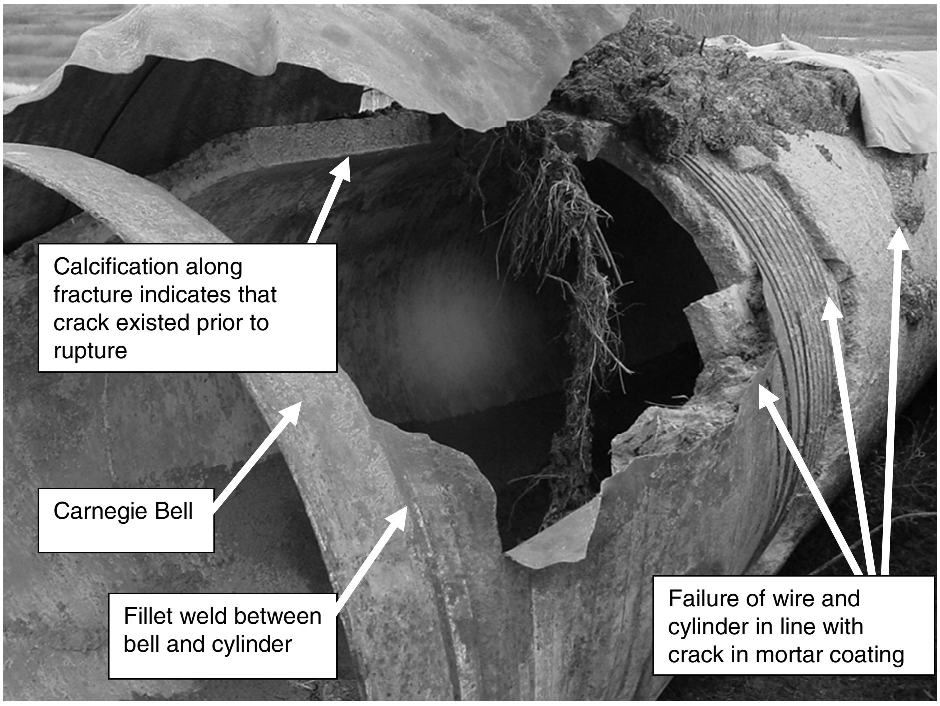
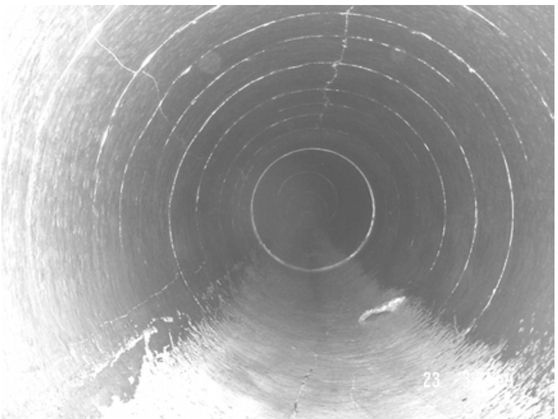
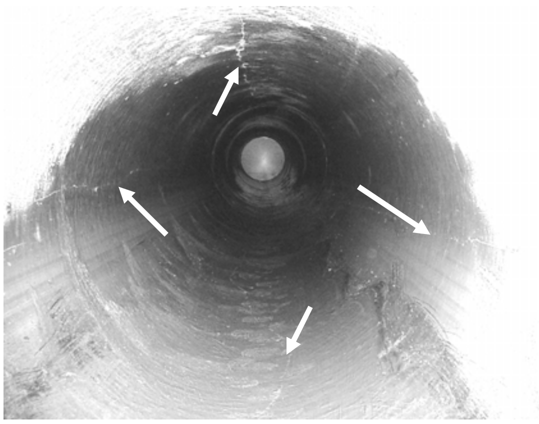


Exhibit 1. Failed pipe section

Immediately following the rupture, LAN performed a visual inspection of the failed water line and testing of mortar and wire samples. Internal observations of the pipe revealed longitudinal cracks along the crown and spring line on both sides of the pipe. Further inspection revealed the internal cracking appeared consistently at invert, crown, and spring lines for a distance of approximately 25,000 linear feet (see Exhibits 2 and 3). The visual inspection was accompanied by sounding to determine if the mortar lining was delaminating from the steel cylinder; however, the lining did not exhibit any noticeable delaminations.



Circumferential cracks



Cracks appeared constantly along crown, invert, and springlines

Exhibits 2. and 3. Interior cracking

As part of the evaluation, several pipe sections were excavated to investigate how the internal cracking correlated to overall pipe condition. Each piece excavated showed signs of external cracking, approximately in line with the internal cracks, indicating the cracks extended through the entire pipe wall.

As the surrounding backfill was removed, large sections of mortar coating separated from the pipe with little or no effort (see Exhibit 4). This delamination resulted in corrosion of both prestressing wire and cylinder. The corrosion likely facilitated the onset of failure. Testing performed on wire samples revealed no signs of hydrogen embrittlement or other obvious manufacturing defects.

Exhibit 4. Delaminated mortar coating

It was determined the repeated rapid closure of an emergency shut-off valve within the plant was the likely cause of the cracking. Hydraulic modeling of this closure scenario performed after the failure indicated that pressures in the pipe may have surged above the maximum design of the pipe for a total pressure of approximately 300 psi. The excessive pressure zone was shown to extend approximately 26,000 linear feet from the pump station, which corresponds well with the visual inspection observations. Internal cracking in the mortar lining was observed consistently for the first 25,000 feet from the pump station. Subsequent to this discovery, the closing rate of the emergency shut-off valve within the plant was modified to reduce the risk of future surges.

Due to the extent of damage observed and risk of collateral damage in the event of a second failure, it was determined the 42-inch water line could not be placed back in service at full pressure without extensive rehabilitation. The potential for further catastrophic failures is likely if the operational pressures exceed the steel cylinder's pressure sustaining capability (determined to be on the order of 30-35 psi).

Because this line is the only source of surface water to the area, groundwater wells had to be brought online and were required to operate at full capacity in order to meet the water demand during the outage. Hydraulic modeling of the area suggested that groundwater wells alone were not sufficient to meet the upcoming summer peak water demands in the area. Therefore, LAN and the City were faced with the task of rehabilitating 26,000 linear feet of 42-inch water line and safely returning it to service before Summer 2005. This schedule left only ten months to complete both design and construction.

Sliplining was deemed to be a viable method to quickly rehabilitate the line in this case due to the construction of the existing pipe. At the time the original 42-inch water line was constructed, there was little development in the area and the result was a relatively flat, consistent vertical and horizontal alignment, with few bends and few interconnections along the alignment.

Leak 3: 48-inch PCCP along Allen Genoa

A water main leak was reported on Allen-Genoa, just south of Queens Road. The leak was visible in the roadway of Allen Genoa, and the source was believed to be a 48-inch diameter prestressed concrete cylinder pipe (PCCP) water line constructed in 1973. The initial observations indicated the possible area of leakage occurred in the immediate vicinity of a corridor of petroleum product pipelines. These lines include a 24-inch, 750 PSI propane gas pipeline, as well as a six-inch natural gas line believed to be located less than one-foot away from the 48-inch water line.

Due to the age of the line and the material of construction, the catastrophic failure mode that can be exhibited by PCCP water lines and the proximity to petroleum pipelines, the City determined it was appropriate to perform an Engineering Assessment in conjunction with repairing the possible leak to determine the condition of the line.

The 16,890 linear foot water line was constructed in 1973 of embedded cylinder PCCP, E-301, 16 gauge steel cylinder, with 6 gauge, class II wire. The joints consist of heavy-walled "Carnegie" bell and spigot rings factory welded to the steel cylinder.

The use of open-cut excavation to determine the nature of the possible leak was determined to be unfeasible due to the proximity of the pipelines. Also, several apartment buildings are located on the southeast corner of Allen-Genoa and Queens Rd., as well as a residential neighborhood located on the southwest corner of the same intersection. As a result, an internal assessment was undertaken.

It was initially expected that the leak was a result of corrosion induced by the cathodic protection system in place on the petroleum pipelines, however, this turned out to be incorrect.

Over a two day period, a total of approximately 1,200 linear feet of the 48-inch water line was assessed. Two leaks were identified within these limits. The leak was evident by an inflow of groundwater at a joint which was only partially grouted, allowing a portion of the steel spigot ring to be exposed. In addition, some of the factory-applied mortar lining was missing in the immediate area of the leak. Severe (Level 3) [5] corrosion was noted at the joint, however the extent of damage was obscured by joint grout and mortar lining, as shown in Exhibit 5 below.

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5 Corrosion Levels

- Level 1 – Minor Surface Corrosion – easy to remove corrosion
- Level 2 – Moderate Corrosion and Pitting – mechanical efforts required to remove corrosion
- Level 3 - Severe Corrosion with Flaking and Major Loss of Metal – removal of corrosion difficult

Exhibit 5: Leak location

As a result of the internal observations, the contractor proceeded to fully expose approximately 12 inches of the steel cylinder adjacent to the area of the leak by removing the mortar lining and the remaining joint grout. After removal of the mortar, it was observed that the steel cylinder at the spigot end exhibited severe corrosion and loss of metal at the weld between the spigot ring and the cylinder. The presence of pinholes was allowing groundwater seepage into the pipe, as shown in Exhibit 6 below.

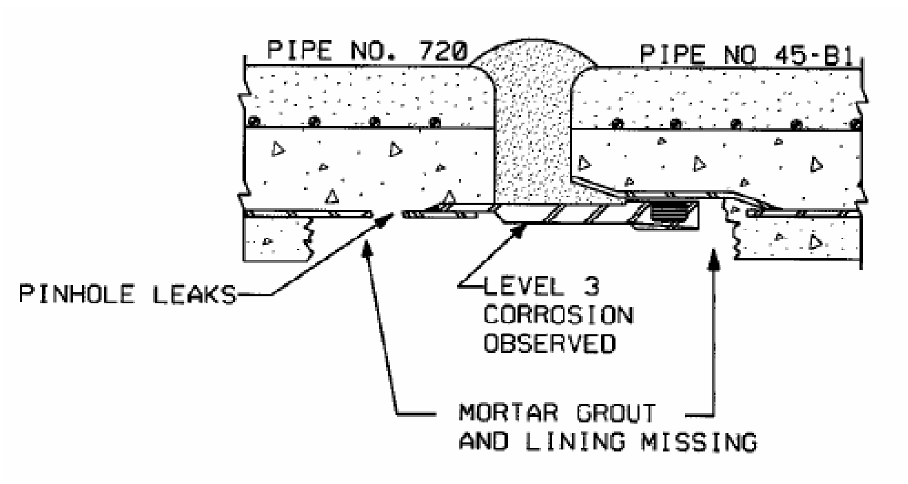

Exhibit 6: Leak location

In accordance with LAN's recommendations, the contractor installed an internal welded butt-strap to encapsulate the severely corroded portion of the steel joint ring and cylinder. A photograph of the completed butt-strap repair prior to replacement of joint grout is shown in Exhibit 7.

Exhibit 7: Completed internal butt-strap repair

In addition to the leaking joint found, the following observations were made:

North of Sta. 117+00

- Approximately 35 joints inspected contained no or insufficient joint grout.
- The existing joint grout that is in place has deteriorated. The grout had the consistency of clay..
- The steel surfaces beneath the clay like grout typically exhibited Level 2 and 3 corrosion.

South of Sta. 117+00

- The joint grout appeared to be intact and in acceptable condition along a majority of the area inspected.

After reviewing the manufacturer's lay schedule for the installation of the water line, it was discovered that Sta. 117+00 was the connecting point for two separate pipe laying crews under the original contract. The constructed segment south of Sta. 117+00 displayed better workmanship of grouting and finishing the pipe joints, while the constructed section north of Sta. 117+00 reflects noticeable lesser quality of mixing and placing grout.

The internal joint grout application and grout materials used failed to adequately pacify the steel surfaces to prevent corrosion. It does not appear that there are any manufacturer related problems with the line, or problems resulting from external sources.

Leak 4: 24- 30- and 36-inch Cast Iron water line along Westheimer, Westpark and Calumet

In the past several years, the City of Houston has been experiencing problems with existing 24, 30, and 36-inch cast iron water lines concentrated in the vicinity of Calumet, Westpark, Westheimer, and Calumet. These water lines were constructed over 50 years ago and consist of cast iron material with lead caulked joints.

The line was hot-tapped in several locations along the length of the line in order to provide coupons for testing. The coupons indicate the existing pipeline is structurally sound, as can be seen in Exhibits 8 and 9.

Coupon removed from the cast iron water line.

Cut-through of bell/spigot joint showing leak caulking

Exhibits 8 and 9: 50+ year old cast iron water line

However, the leak history of these lines indicates that a disproportionate amount of effort is required by City forces to maintain these lines making them a maintenance burden for the City.

Interviews with City maintenance crews indicate that all of the recent repairs have been located at joints, where the existing lead caulking has failed. Since the line is constructed along major thoroughfares within the Central Business District and the Galleria area in Houston, it was not feasible or cost-effective to continue to excavate each joint to make repairs, or to construct a separate parallel large diameter line.

The chosen rehabilitation was to install internal joint seals, requiring only periodic access pits to be excavated.

Prioritization Condition Assessment of Other Large Diameter Water Mains

The failure that occurred on the 60-inch PCCP water line was one of the first major catastrophic failures with which Houston has dealt. However, based on experience from other cities, PCCP manufactured in the late 1960's and 1970's is suspect for development of embrittlement due to the fact that rules regulating the manufacture of wire during this period may not have adequately protected the wire from dynamic strain aging. Since many of the quality control regulations, which make recent pipe less susceptible to hydrogen embrittlement, were not implemented until the late 1980's, the problems encountered in this 60-inch water line may exist even in later projects.

The City of Houston currently maintains approximately 1,000,000 linear feet of large diameter PCCP, ranging from 24-inch to 108-inch. Approximately 400,000 linear feet were manufactured in the most suspect range from 1968 to 1978. There are also miles of 50+ year old cast iron lines in the city's system.

As a result, the City is now taking a more proactive approach to avoid similar failures through a prioritized assessment and maintenance program. The first step in this program is to prioritize existing large diameter lines in the system for future assessment on the basis of the following major categories:

- Age
- Repair history
- Critical nature of line; determine if alternate water sources exist, and the impact to system pressures if the line were lost
- Risk of collateral damage (proximity to major thoroughfares, schools, hospitals, residential neighborhoods and other critical infrastructure)
- Type of pipe (material, joint type, prestressing wire class)
- Cathodic Protection system (Magnesium anodes, impressed current, affect of stray currents from adjacent systems)
- Operational and pressure history
- Review of construction inspection records
- Adjacent construction projects

Record information, including pipe design, cathodic protection system records, repair history, valve locations (noted as working or non-working), and customers affected by shut downs will be compiled and stored in a searchable GIS mapping system for future use.

In order for a complete prioritization to be performed, the following Tasks must be performed:

1. A thorough document search should be performed to retrieve record information owner and pipe manufacturer. Design drawings, record drawings, and lay schedules should be gathered from each project on the list.
2. A preliminary prioritization list should be developed using rehabilitation history information from available repair history data and interviews with maintenance or operations personnel. This recent rehabilitation information will provide a better understanding of the condition of each line.
3. Determine customers served by the lines, and if alternate sources of water exist.
4. The condition of isolation valves along the critical lines and connections should also be assessed. Record research and physical test cuts should be performed to determine the best locations to isolate line sections for assessment and, in the case of an emergency, to reduce the number of customers put out of service.
5. Other construction in the area that may have impacted the line in the past.
6. Monitoring to determine the state of the existing cathodic protection system. Locations of existing anode beds can be compared to rehabilitation locations.
7. Materials testing to examine soil, mortar, wire and cylinder properties.
8. If the line is accessible, perform visual and electromagnetic inspections to establish a baseline pipe condition.
9. Long-term acoustic monitoring to evaluate the potential rate of deterioration.

As demonstrated by these rehabilitations, there is significant cost associated with physical testing and upgrades necessary to support the inspection and rehabilitation process. Therefore, prioritizing lines can help to schedule those improvements BEFORE a catastrophic failure.

ESTABLISHING A CCTV INSPECTION, ASSESSMENT AND IMPROVEMENT PROGRAM FOR A LARGE DIAMETER COLLECTION SYSTEM

by
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BACKGROUND

The Inland Empire Utilities Agency (IEUA), formed in 1950, serves a 242 square mile area located in the southwest corner of San Bernardino County, approximately 35 miles east of Los Angeles. IEUA provides regional wastewater service and imported water deliveries to several contracting agencies. IEUA's Non-Reclaimable Wastewater System (NRWS) is a large diameter pipeline and force main system, about 55 miles long, conveying non-reclaimable wastewater or "brine." Over the years, records for this system became outdated and the condition of the pipelines and manholes were unknown.

In the summer of 2005, IEUA contracted with PBS&J, an engineering consulting firm, to locate, inspect and assess the NRWS, including approximately 5 miles of force main, and 613 manholes. PBS&J teamed with Houston & Harris, PCS, Inc. (H&H) to inspect the sewers. Cecilia's Safety Services prepared the temporary traffic control plans, acquired the traffic permits, and performed the traffic control set up and take down for the inspection efforts. California Surveying Corporation located the inspected manholes using global positioning system (GPS) equipment to locate the manholes to the nearest 0.1 of a foot.

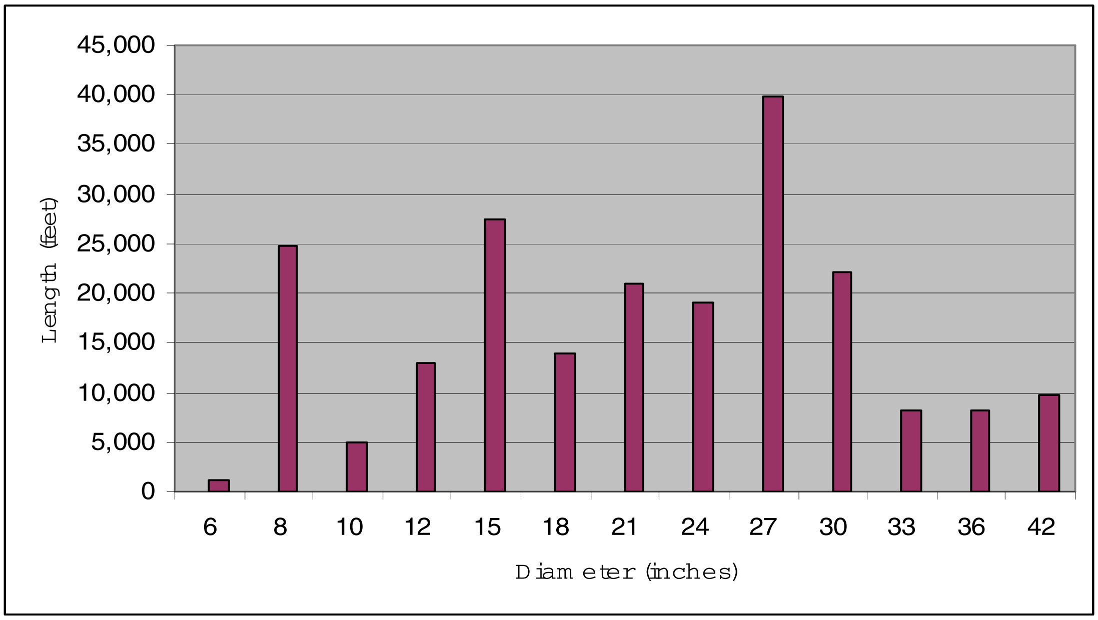
The NRWS Capital Improvements Program Plan provided IEUA with recommendations for capital improvements, cleaning recommendations and on-going system maintenance efforts for the NRWS. The inspection and assessment work occurred between September 2005 and January 2006. The fast-paced schedule required the completion of the final report by March 1, 2006.

DETAILS OF THE NON RECLAIMABLE WASTEWATER SYSTEM

The NRWS collects industrial flows from businesses located within the IEUA service area. The flows discharged into the system may contain hazardous constituents and often is high in suspended solids. These materials, while not always harmful to the pipelines, tend to accumulate and constrict the flow as well as make the wastewater non-reclaimable for beneficial uses.

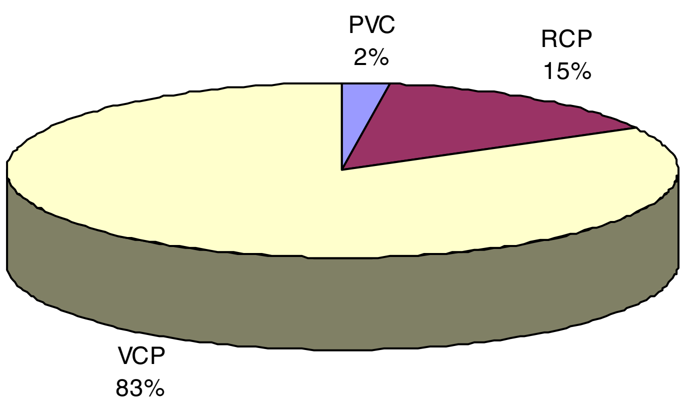
The NRWS system ranges in diameter from 6 inches up to 42 inches. Based on an inspected length of approximately 255,374 feet, Figure 1 shows the length of pipe for each diameter in the NRWS.

Figure 1- Distribution of Pipe Diameters



The pipeline material is mostly vitrified clay pipe (VCP), while unlined reinforced concrete (RCP) pipe and polyvinyl chloride (PVC) make up the remainder of the system. Figure 2 shows the amount of material in each category, by length.

Figure 2 -Pipe Material



INSPECTION CRITERIA AND PROCEDURES

Closed circuit television cameras offer valuable insight to the internal structural condition of buried infrastructure. Video inspection of sewer pipelines is used to evaluate the existence and severity of cracks, misaligned joints, and potential sources of infiltration. Analysis of this data determines where rehabilitation is required and which method of repair is most appropriate and cost effective.

Uniformity and consistency of the observation comments is paramount in providing useful assessment results. Using standardized inspection observation codes, different CCTV camera operators can provide consistent comments about the condition of the pipe and assign uniform values to each observation based on severity.

The IEUA established its own observation and defect codes based on the Water Resource Centre (WRc) codes. Before beginning the inspections, the operators reviewed the codes to become familiar with them, and clarified uncertainties with IEUA staff before beginning the inspections. These codes are compatible with the National Association of Sewer Service Companies (NASSCO) standards for pipeline assessment.

INSPECTING THE NRWS PIPELINES

Inspection and assessment of sanitary sewer pipelines and manholes is an important part of maintaining and operating a sewer system. Inspection and assessment is used to evaluate the existing system conditions and potential defects, which may contribute to potential overflows. Such conditions include root intrusion at misaligned joints or cracks and inflow and infiltration entering into the system through cracks in pipes, manholes or via illegal storm drain connections. The results of a sewer assessment are then used to determine the funding required to repair, rehabilitate and replace an aging collection system and to prioritize how the funds should be allocated.

Our team began with outdated as-builts, incomplete electronic records for the manholes and pipelines, and a reliance on the institutional knowledge of the operations staff. Plus, IEUA, for the first time, specified digital video and database submissions for the inspection and assessment work, which created new electronic data management and storage challenges for IEUA. IEUA developed an asset numbering scheme to facilitate incorporating the collected sewer system data into IEUA's GIS system. IEUA required that the manholes be located by coordinates and the data provided for GIS mapping.

A challenging task involved the force main inspection. Locating the access points to the force mains required as-built record reviews, field reconnaissance, pot holing, and some excavation to locate the cleanouts. Additionally, concerns about the by-pass pumping process and wet well detention time were addressed.

H&H inserted closed circuit television (CCTV) cameras into the sewer pipelines, and, by remotely controlling the cameras, recorded the condition of the interior of the pipelines. Ultimately four (4) inspection vehicles were on the job site to maintain the aggressive schedule. PBS&J and H&H also inspected the associated manholes. PBS&J recorded the physical condition of the structures, while H&H captured digital images of the manholes.

PIPELINE RENEWAL AND REHABILITATION OPTIONS

Replacement of existing systems can be very disruptive to a community. In many cases, the problem sewer may have isolated or point defects that can easily be repaired. It is becoming more common to rehabilitate sewers instead of constructing new replacement or relief sewers. Alternative construction techniques, such as lining, can significantly reduce community impacts and provide a cost effective option to replacement. Of course, there are cases where due to the age of the sewer and the extent of defects, there may be no alternative but to replace the sewer line.

Certain rehabilitation methods can be performed quickly and with little impact to the community at a fraction of the cost of replacing a facility, but the ability to maintain rehabilitated facilities, along with the estimated benefits, must be considered. The approach to selecting appropriate rehabilitation alternatives are summarized below in Table 1. As a reminder, the selection of a method based on the general rules below is not a substitute for the experience and knowledge of the staff and engineers performing the assessment. The unique situation and condition of each segment must be considered when finalizing recommendations. As such, not every recommendation will rigidly adhere to the methods and rules below.

Table 1 - Sewer Improvement Alternatives

Description of Defect	Recommended Method
Roots, broken or cracked pipe, misaligned or open joint, and/or grade break at 1 or 2 locations along a pipe	Point repair (if total pipeline length is > 300 feet); Replacement (if total pipeline length is < 300 feet)
Roots only, more than 2 locations	Lining, Chemical Herbicide or Repetitive Cleaning
Multiple cracks only – minor to medium	Lining
Roots most joints with multiple cracks, no offsets	Lining
Roots at most joints with major offsets, breaks, major cracks or major grade breaks and all other conditions	Replacement
Severe defects requiring replacement in difficult to access locations or areas of high traffic congestion	Pipe bursting, bore and jack, directional drilling or micro tunneling

The following descriptions summarize a variety of rehabilitation and repair methods considered for the NRWS.

Point Repairs

Where defects were noted from the video inspections at one to two locations between manholes, point repairs were recommended. However, for short pipe segments less than 300 feet with two defects, point repairs may affect the integrity of the remaining pipe and full pipe replacement is usually recommended. Point repairs are cost effective alternatives to full pipe replacement if the remaining portions of the pipe are in good condition as noted from video inspections and replacement of the entire pipe segment appears unwarranted.

Pipe Lining

Pipeline lining offers a trenchless method to rehabilitate deteriorating sewer mains. Because no open trenching is required during the lining application, there is minimal disruption to residents and business owners during construction. Lining of sewer mains is recommended for pipes with rooting problems and/or cracks with no major offset joints.

Where breaks, major offsets, or bends are noted in the video inspections, lining is not recommended because the lining material cannot be applied properly where these conditions occur. In choosing an appropriate repair method, there are cases where it may be cost effective to consider combining point repairs with lining.

Several methods of lining can be used to rehabilitate a pipeline. Cured-in-Place Pipe (CIPP) lining involves inverting a resin-impregnated fabric tube into a clean, existing pipeline, then curing it in place with hot water or steam. Fold-and-form lining requires pulling a fabric tube, saturated with a resin and folded into a U-shape, into a clean, existing pipe, then expanding the tube to the shape of the pipe with pressurized water before curing with heated water or steam. Both of these methods require the flow to be plugged or diverted during the installation process.

Spiral wound pipe is another lining method that inserts a continuous strip of reinforced plastic, usually 6 or 8 inches wide, into an existing pipe, which interlocks to form a watertight seal. Spiral wound pipe can typically be installed in a live sewer without plugging or diverted the flow.

All of these lining methods can provide a system with independent structural integrity that does not rely on the host pipe for strength.

Pipe Bursting

Pipe bursting offers a solution to replacing defective pipe that has access at its end points but not between, such as pipe that crosses a freeway or major intersection.

Pipes that are good pipe bursting candidates generally have no bends or curves and have adequate slope with very minor or no sags. Access pits are required at the ends of the pipe segment being burst. If any service connections must be reinstated, access pits are required at these locations as well. The size of the pipe can generally be increased to the next standard pipe diameter above the existing diameter, and possibly to the next standard pipe size depending on the existing soil conditions and adjacent utilities. The material used for the replacement pipe includes high density poly-ethylene (HDPE) pipe or joint-fused PVC pipe. This method may be more costly than traditional dig and replace, but it is generally faster and less disruptive to the environment and community, thereby resulting in intangible benefits.

Easement Access

Sewer mains located in some easements are a concern due to inadequate access to maintain these areas and to replace with traditional dig and replace methods. Sewer mains in easements with poor or no access for maintenance or emergency repair work often result in higher repair costs due to using specialty access equipment, hand labor, and mitigation of unplanned environmental impacts. Conversely, sewer mains with appropriate access, such as access paths or in parking lots, would not necessarily need relocation. Rather, coordination with the land owner may be all that is required. Another option is using carrier pipes or casings in the easements, so that sewer mains can be replaced outside the constrained easement areas. Private pumping is also an option. In most cases, the topography of the area will dictate the alignment of the new gravity sewer system. As such, relocating easement sewers may not be cost effective.

Often the cost of relocating sewers is expensive because of the costs associated with constructing new facilities, as well as abandoning the old facilities. If the annual costs of maintaining an aging easement sewer, plus the benefits to the community and environment, outweigh the capital expense to relocate, it may be prudent to relocate easement sewers. However, if operations staff can adequately access the sewer, and the facility is in good condition or easily improved, the sewer does not need relocating. To properly evaluate the feasibility of realigning pipelines, with poor or no access, into the public streets and out of private property, an agency should conduct, on a case-by-case basis, re-direction studies to consider alternatives and recommend accessibility improvements and costs.

Pipeline and Manhole Replacement

Replacements of pipe segments are recommended where multiple occurrences of breaks, cracks, misaligned joints, grade breaks and roots are identified in the video inspections. In addition, where pipe replacement is recommended, replacement of both manholes at each end of the pipe is also recommended. For maintenance purposes, pipelines with a diameter of 6 inches or less are recommended for replacement with 8 inch diameter pipe. Actual replacement must be evaluated on a case by case basis after considering several factors such as pipe condition, slope, cleansing velocity, capacity, and available maintenance equipment and methods.

ASSESSMENT CRITERIA AND PROCEDURES

Several factors set the priority of projects identified during the assessment process, although the condition of the pipe is usually the primary factor. Other factors can include a goal to reduce sanitary sewer overflows, reduce infiltration and inflow in pipes located below the water table, or reduce maintenance efforts by improving the pipe. Other considerations include coordinating surface and utility improvements with the agency or municipality having jurisdiction over the property through which the sewer facilities pass, and minimizing community disruption.

Each NRWS segment inspected was ranked using a scale of 1 through 5 to indicate the severity of pipeline's condition, with 5 being the worst condition. Table 2 shows the general condition ranking associated with each severity level as well as the recommended response time to complete the recommended action.

Table 2 - Condition Severity Ranking

1	2	3	4	5
Good	Adequate	Moderate	Poor	Failing
Maintenance	5 + years	3 to 5 years	1 to 2 years	Immediate

The severity assigned to each pipeline is based on the criteria for each observation listed in Table 3. Using the observation and severity, staff then made a preliminary recommendation for each pipeline segment. Table 4 summarizes the typical preliminary recommendations for each observation and severity ranking. The ranking prioritized each segment from best to worst and estimated the useful life remaining for each. Every pipeline also received one of the following recommended actions: 'no action,' 'clean/maintain,' 'point repair,' 'rehabilitate,' 'replace,' or 'evaluate/re-inspect.'

Many benefits stem from performing a systematic assessment of a sanitary sewer system. Some of the benefits realized include:

- Providing a comprehensive overview of the system's condition;
- Prioritizing the pipe from worst to best, thereby achieving the most benefit for each dollar spent by working on the worst facilities first;
- Estimating the projected costs to complete the identified work;
- Normalizing the pace of improvement work to avoid large fluctuations in the capital improvements program from year to year;
- Reducing emergency responses as the system is improved; and
- Highlighting common defects so that the agency can address problems on a global level (e.g. focusing on a specific material that may fail more frequently

than others, or showing high grease concentrations in areas that may need a grease control program).

Table 3 - Infrastructure Assessment Criteria

Observation	SEVERITY				
	1	2	3	4	5
Cracks • Circular • Longitudinal • Multiple	None	Very small Hair line crack(s)	Hair line crack(s) <50% of ID in length	Cracks ≤1/8" wide or >50% of ID in length	Cracks >1/8" wide
Broken Pipe	None	Connecting cracks, no displacement	Connecting cracks, displacement ≤¼"	Connecting cracks, displacement >¼"	Collapsed pipe, impassable
Joints - Offset	Minimal	Up to ½ of the pipe thickness	½ to thickness of the pipe	Thickness of the pipe to 1 ½ times	Over 1 ½ times the thickness of the pipe
Joints – Separation	None	Gasket exposed	Bell exposed	Dirt exposed at top	Dirt exposed at invert
Roots	Minimal	10% to 35% Fine roots	35% to 60% Fine/medium roots	60% to 80% Medium roots	80% to 100% Tap root(s) visible
Grease	None	≤¼" thick	¼" to ½" thick	½" to 2" thick	>2" thick
Debris Accumulation	Minimal	Sporadic deposits (no rocks)	≤10% of ID (no rocks)	10% to 25% of ID and/or rocks	>25% of ID or impassable
Erosion (Typ Conc Pipe)	None	Rough surface	Exposed aggregate	Exposed rebar	Missing concrete
Corrosion (metal pipe only)	None	Minimal	Light tuberculation	Moderate tuberculation	Impassable, heavy tuberculation
Mineral Deposits	None	Minimal (possible infiltration)	≤10% ID thickness	>10% ID thickness	Impassable, heavy mineral deposits
Infiltration	None	Dripping	seeping	Constant stream	Gushing water
Sag	None	Minimal (probably not perceptible)	≤25% of ID	25% to 75% of ID	>75% of ID
Flow Capacity	Minimal	2/5 or less full	2/5 to ½ full	½ to ¾ full	¾ to completely full

Table 4 - Pipeline Recommendation Criteria

Observation	SEVERITY				
	1	2	3	4	5
Cracks • Circular • Longitudinal • Multiple	No Action	No Action or Rehabilitate	No Action or Rehabilitate	Rehabilitate	Rehabilitate or Replace
Broken Pipe	No Action	No Action or Rehabilitate	Point Repair or Rehabilitate /Replace	Point Repair or Replace	Immediate Point Repair
Joints - Offset	No Action	No Action or Rehabilitate	Point Repair and/or Rehabilitate	Point Repair and/or Rehabilitate /Replace	Point Repair and/or Rehabilitate /Replace
Joints – Separation	No Action	Rehabilitate	Rehabilitate	Point Repair and/or Rehabilitate /Replace	Rehabilitate or Replace
Roots	No Action	Clean and Rehabilitate	Clean and Rehabilitate	Clean and Rehabilitate	Clean and Rehabilitate /Replace
Grease	No Action	Clean	Clean	Clean	Clean
Debris Accumulation	No Action	Clean	Clean	Clean	Clean
Erosion (Typ Concrete Pipe)	No Action	Rehabilitate	Rehabilitate or Replace	Rehabilitate or Replace	Replace
Corrosion (metal pipe only)	No Action	Ream and Rehabilitate	Ream and Rehabilitate	Replace	Replace
Mineral Deposits	No Action	No Action or Rehabilitate	Point Repair or Rehabilitate	Rehabilitate	Rehabilitate
Infiltration	No Action	No Action or Rehabilitate	Point Repair or Rehabilitate	Rehabilitate	Rehabilitate
Sag	No Action	No Action	Any option	Replace	Replace
Flow Capacity	No Action	No Action	No Action	Evaluate capacity	Evaluate capacity

RESULTS AND RECOMMENDATIONS

Overall, the pipelines that were inspected were in generally good condition. Nearly 21 percent of the segments require no action. However, the system needs cleaning. Half of the inspected segments have debris accumulation or deposits in the pipeline. Structural defects were found in approximately 17 percent of the system, requiring repairs, rehabilitation or replacement. Less than 10 percent of the system should be re-inspected due to excessive debris build up or inaccessibility that prevented a complete inspection from being performed. Table 5 summarizes the inspection and assessment results.

Table 5 - Pipeline Inspection and Assessment Results by Length

Action	Feet	Miles	Percentage
No Action	54,636	10.35	21%
Maintenance	133,558	25.30	53%
Repair/Rehabilitate/Replace	43,779	8.29	17%
Re-Inspect	23,401	4.43	9%
TOTALS	**255,374**	**48.36**	**100%**

The recommended Capital Improvement Program identified projects to be completed beginning July 1, 2006. A five-year and a ten-year CIP were developed. The relatively good condition of the system allowed the improvements to be spread over a ten year period, reducing the annual capital expenditure. However, the overall ten-year CIP is more expensive than the five-year CIP because of inflation, estimated at four percent per year. Also, additional capital projects not yet identified may arise as the system's useful life decreases with time.

PBS&J also recommended an aggressive cleaning program to clean every segment in the NRWS over a three-year period. This cleaning effort will ensure that every pipe segment and manhole is properly maintained before implementing a recommended five-year cleaning and maintenance program. The 5 year program should be designed to clean every pipe at least once every five years, and to clean and maintain other segments more frequently as determined by the circumstances.

Additionally, IEUA should implement a routine inspection and assessment program to ensure that every pipe is inspected at least once every seven years. The inspection information will provide a basis for estimating the remaining life of the NRWS as well as identify areas requiring improvements before catastrophic failure and/or a sanitary sewer overflow occurs.

Other studies recommended to improve the operations and capacity of the NRWS include an Inflow and Infiltration (I/I) Study to identify and correct I/I problems that reduce capacity and generate higher billings, and a Hydraulic Modeling Study, which will provide a dynamic model of the system and forecast when the system will exceed its current capacity.

SCATTERGRAPH PRINCIPLES AND PRACTICE
Characterization of Sanitary Sewer and Combined Sewer Overflows

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Abstract

Sewer overflows pose a significant threat to public health and the environment, contributing to beach closures, contamination of drinking water, and other concerns. Knowing when and where they occur – as well as their duration, volume, and frequency – are important pieces of information needed to assess their impact and minimize their future occurrence.

Sewer overflows are readily identified by evaluating flow monitoring data on a scattergraph. Practical examples from flow monitoring locations throughout the United States are provided, demonstrating the scattergraph signatures of sanitary sewer overflows (SSOs) and combined sewer overflows (CSOs) under various conditions. Techniques are also developed to estimate their duration and volume from flow monitor data.

Introduction

Sewer overflows pose a significant threat to public health and the environment, contributing to beach closures, contamination of drinking water, and other concerns. Knowing when and where they occur – as well as their duration, volume, and frequency – are important pieces of information needed to assess their impact and minimize their future occurrence.

Sewer overflows are readily identified by evaluating flow monitoring data on a scattergraph. The scattergraph is a graphical tool that displays flow depth and velocity data from a sewer flow monitor. The resulting patterns form characteristic signatures that provide insight into conditions within a sewer (Enfinger & Keefe 2002). Scattergraph signatures for sewer overflows have been previously reported in the literature (Stevens & Sands 1995). The scattergraph signatures of sanitary sewer overflows (SSOs) and combined sewer overflows (CSOs) are further discussed in this paper, along with techniques to estimate their duration and volume from flow monitor data.

Sanitary Sewer Overflows

A sanitary sewer overflow (SSO) is a discharge of untreated wastewater from a sanitary sewer system. According to the Environmental Protection Agency (EPA), SSOs are caused by a variety of reasons – including inadequate sewer design and construction, insufficient operation and maintenance, power failures, and vandalism (EPA 2004). These situations are often compounded by infiltration and inflow – contributing to an increase in the duration, volume, and/or frequency of overflow events.

Regardless of the contributing factors, sewers often experience a common sequence of hydraulic events prior to an SSO – including uniform flow, backwater, and surcharge conditions. Profile views depicting this sequence are provided in Figure 1.

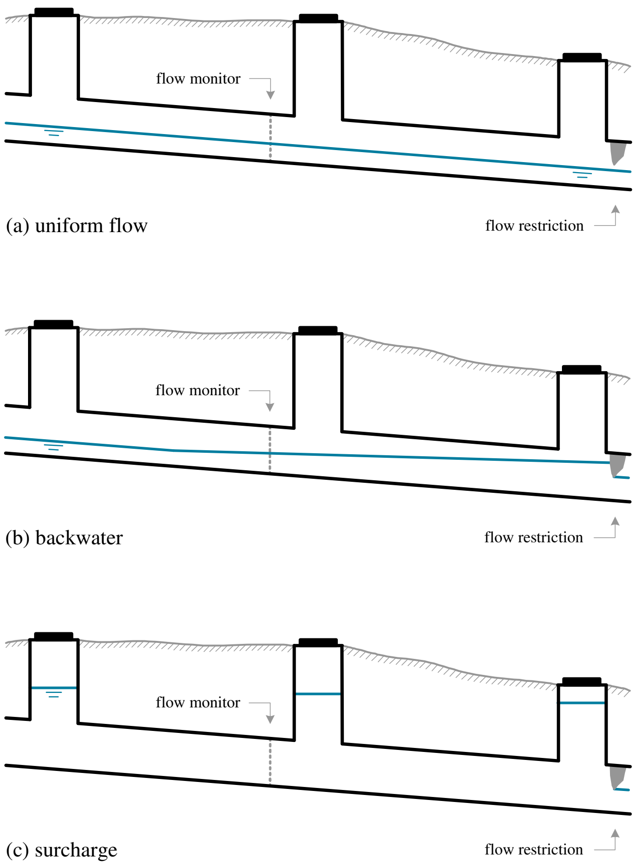


FIGURE 1: Prelude to an SSO — a Profile View

Uniform flow conditions are often assumed in a sewer under normal flow conditions, as shown in Figure 1(a). Although a flow restriction is shown in this example, flow conditions are not adversely affected at lower flow depths. However, as the flow depth increases, the flow restriction throttles flow through the sewer, resulting in backwater and surcharge conditions, as shown in Figures 1(b) and 1(c), respectively. The operational capacity of this sewer is less than its intended design capacity, and once the surcharge depth reaches the rim elevation of a nearby manhole, an SSO occurs.

Note the location of the flow monitor relative to the flow restriction in Figure 1. The flow restriction is located downstream from the flow monitor. As a result, the sequence of hydraulic events that leads to an SSO leaves a distinct pattern that can be identified on a scattergraph of flow depth and velocity data, as shown in Figure 2.

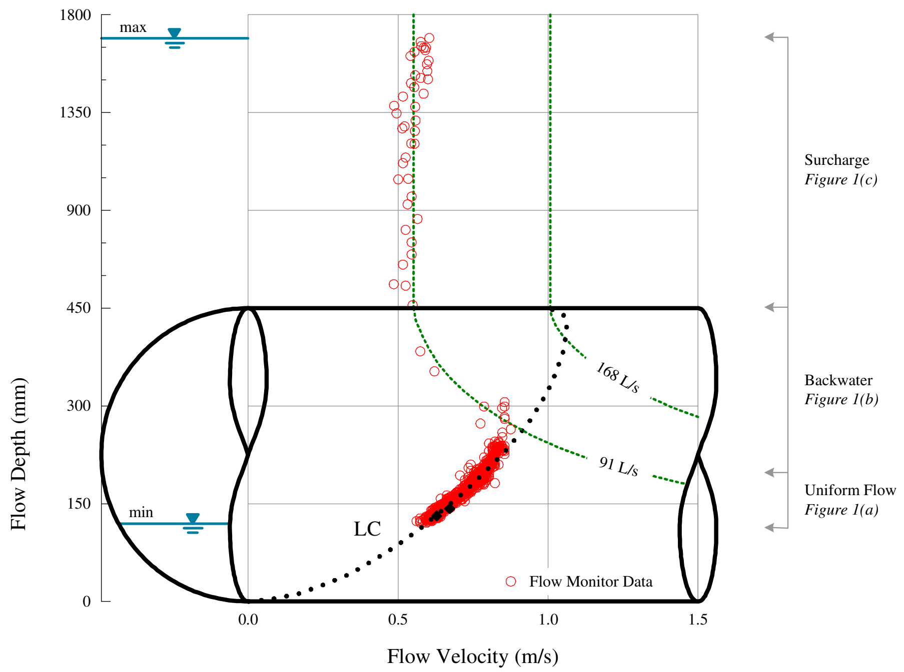


FIGURE 2: Prelude to an SSO — a Scattergraph View

During uniform flow conditions, the relationship between flow depth and velocity is described by the Manning Equation (Enfinger & Kimbrough 2002). This equation is depicted by the *pipe curve* shown in Figure 2. Uniform flow conditions are identified on a scattergraph when the flow monitoring data are consistent with the *pipe curve*. However, as backwater conditions develop, flow conditions become deeper and slower and are revealed on the scattergraph as a departure from the *pipe curve*. The flow rate at which this occurs is noted by an iso-Q™ line and represents an operational capacity that is only 54% of the expected capacity of this sewer (Enfinger & Stevens 2006).

The scattergraph signatures of SSOs under various conditions are discussed in the following sections. Despite the variations, the common sequence of hydraulic events shown in Figure 2 is noted in each case.

Sanitary Sewer Overflow (Upstream)

The scattergraph signature of an SSO depends on the type of overflow and its position relative to a flow monitor. A profile view of an SSO that occurs upstream from a flow monitor is shown in Figure 3.

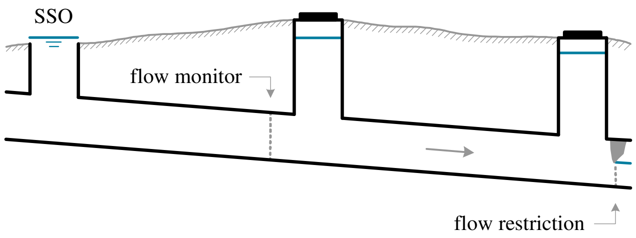


FIGURE 3: Upstream SSO

This SSO is identified on a scattergraph by a cluster of surcharge data points at a constant flow depth and a constant velocity, as shown in Figure 4. The depth reported by the flow monitor during the SSO is controlled by the overflow elevation, and the velocity is controlled by the operational capacity of the downstream sewer.

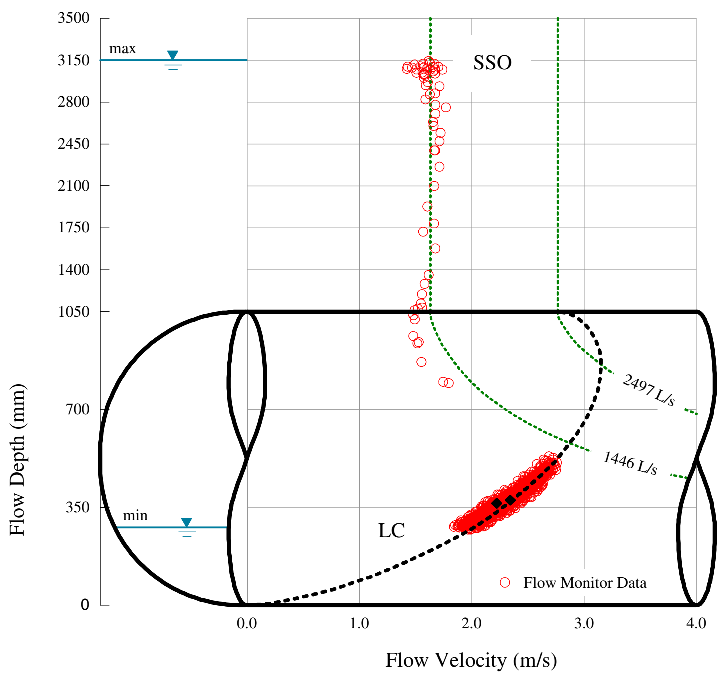


FIGURE 4: Scattergraph of an Upstream SSO

Based on the scattergraph, the operational capacity of this sewer is 1446 L/s – only 58% of the expected capacity under uniform flow conditions. Surcharge conditions are observed up to a flow depth of nearly 3150 mm when an SSO occurs upstream from the flow monitor.

Sanitary Sewer Overflow (Downstream)

The scattergraph signature of a downstream SSO can also be identified. A profile view of an SSO that occurs downstream from a flow monitor is shown in Figure 5.

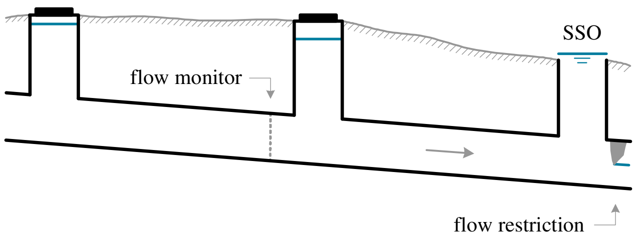


FIGURE 5: Downstream SSO

Both upstream and downstream SSOs are characterized by a constant flow depth during an overflow. However, the additional flow escaping the system during a downstream SSO is detected by the flow monitor as an increase in velocity during the overflow event, as shown in Figure 6.

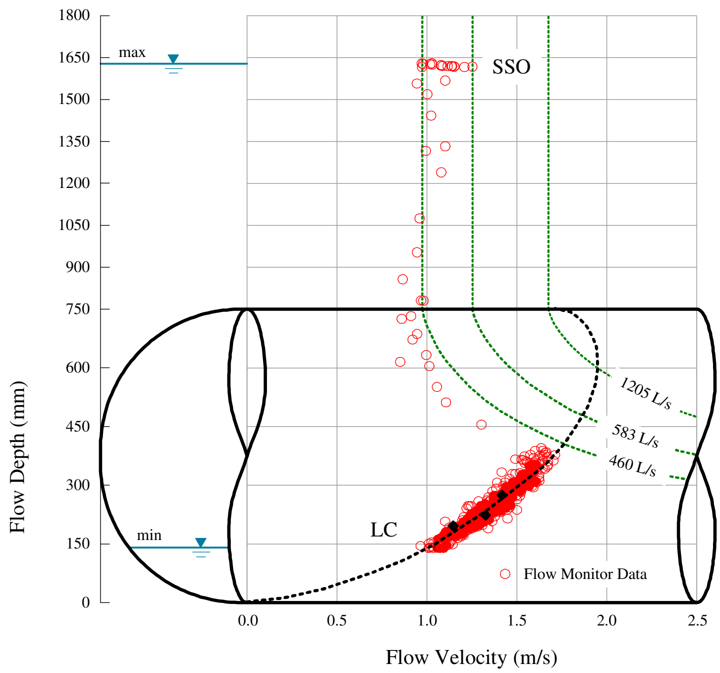


FIGURE 6: Scattergraph of a Downstream SSO

Based on the scattergraph, the operational capacity of this sewer is about 460 L/s – only 38% of the expected capacity under uniform flow conditions. Surcharge conditions are observed up to a flow depth of nearly 1650 mm when an SSO occurs downstream from the flow monitor. The maximum overflow rate is determined using iso-Q lines and is approximately 123 L/s (583 L/s – 460 L/s).

Sanitary Sewer Overflow from an Overflow Pipe

Some municipalities have SSOs from fixed points within the sewer system that overflow to a storm sewer or directly to receiving waters (EPA 2004). A profile view of an SSO that occurs from an overflow pipe is shown in Figure 7.

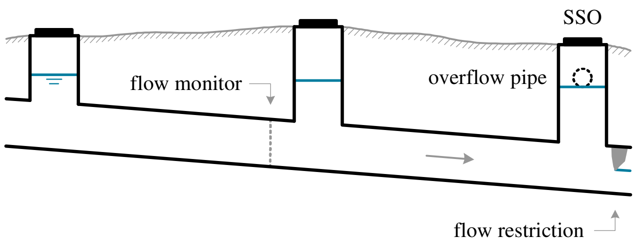


FIGURE 7: SSO from Overflow Pipe

The scattergraph shown in Figure 8 is from a 250-mm sewer equipped with a 200-mm overflow pipe located in a manhole downstream from the flow monitor.

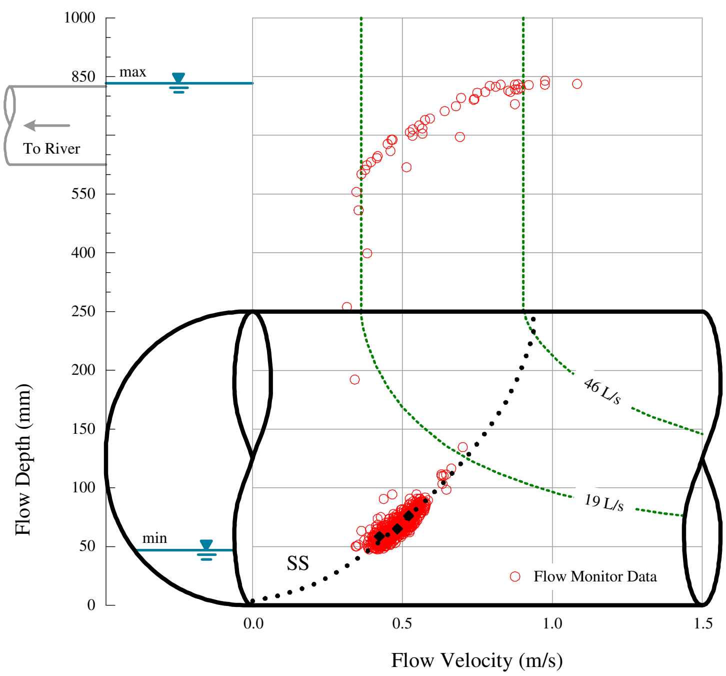


FIGURE 8: Scattergraph of an SSO from an Overflow Pipe

Based on the scattergraph, the operational capacity of this sewer is 19 L/s – only 41% of the expected capacity under uniform flow conditions. The SSO is activated at a flow depth of 625 mm – the invert elevation of the 200-mm overflow pipe.

Reverse Flow

Reverse flow in a sewer system is rare but can occur in certain situations. A profile view of reverse flow is shown in Figure 9.

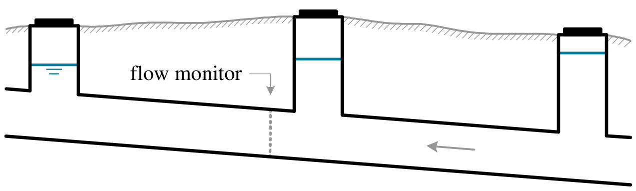


FIGURE 9: Reverse Flow

The scattergraph shown in Figure 10 is from a 300-mm sewer that is *overpowered* by a much larger downstream interceptor.

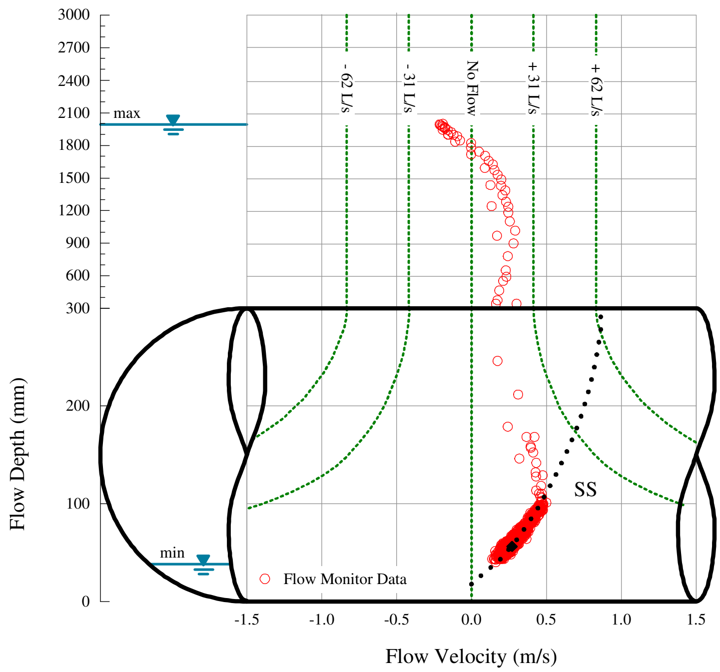


FIGURE 10: Scattergraph of Reverse Flow

During a rain event, this sewer experiences backwater and surcharge conditions. However, note the sequence of events that occurs during surcharge conditions. The flow rate begins to slow down at 1000 mm and eventually comes to a momentary stop at 1800 mm. Reverse flow is observed above this depth and may have led to an SSO.

Combined Sewer Overflows

A combined sewer overflow (CSO) is a discharge of untreated wastewater from a combined sewer system, most often occurring at a CSO regulator designed for this purpose. CSOs generally occur during wet weather events, when the combined flow rate of wastewater and storm water exceeds the capacity that a regulator structure is configured to convey to the WWTP (EPA 2004). Excess flows are discharged to receiving waters.

Most combined sewers experience a common sequence of hydraulic events prior to a CSO, based on the design of the CSO regulator. Those equipped with end weirs or side weirs often generate backwater conditions in the incoming sewer, and once the backwater depth reaches the weir elevation, a CSO occurs. This sequence of hydraulic events leaves a distinct pattern that can be identified on a scattergraph of flow depth and velocity data. The scattergraph signatures of CSOs from regulator structures equipped with end weirs or side weirs are discussed in the following sections.

Combined Sewer Overflow (End Weir)

Some CSO regulators are equipped with an end weir that is constructed perpendicular to the incoming wastewater flow. Dry weather flow is diverted to the WWTP. However, once the flow depth exceeds the weir height, additional flow is carried over the weir and is discharged to the receiving water. A plan view of a CSO regulator equipped with an end weir is shown in Figure 11.

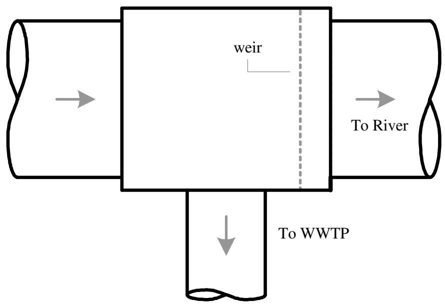


FIGURE 11: Plan View of a CSO Regulator with an End Weir

The scattergraph shown in Figure 12 displays data from a flow monitor installed in a 750-mm sewer located just upstream from a CSO regulator equipped with an end weir.

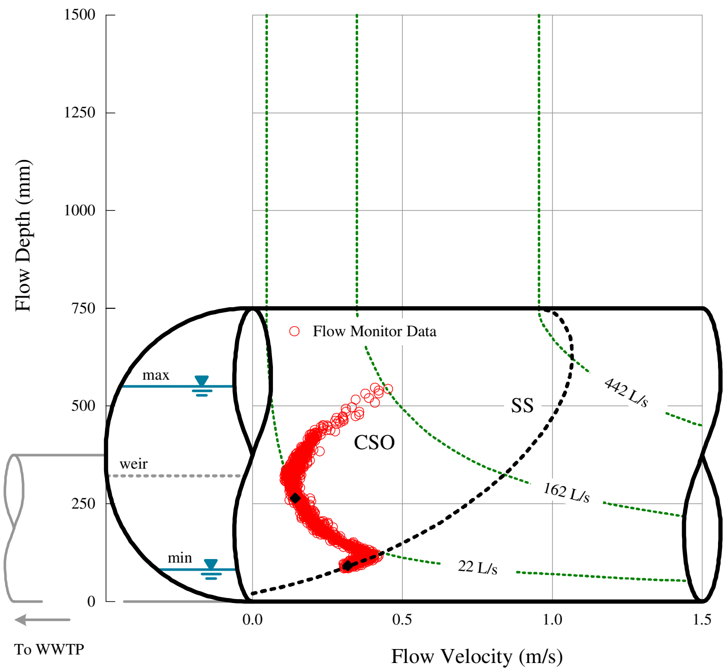


FIGURE 12: Scattergraph of a CSO from an End Weir

Dry weather flows are diverted to the WWTP through a 375-mm sewer. Based on the scattergraph, the 375-mm sewer conveys flows up to 22 L/s to the WWTP, and the CSO is activated at a flow depth of 325 mm. The maximum overflow rate is determined using iso-Q lines and is approximately 140 L/s (162 L/s – 22 L/s).

Combined Sewer Overflow (Side Weir)

Some CSO regulators are equipped with a side weir that is constructed along the side of the incoming sewer. Dry weather flows are funneled into a smaller sewer and continue to the WWTP. However, once the flow depth exceeds the weir height, additional flow is carried over the side weir and is discharged to the receiving water. A plan view of a CSO regulator equipped with a side weir is shown in Figure 13.

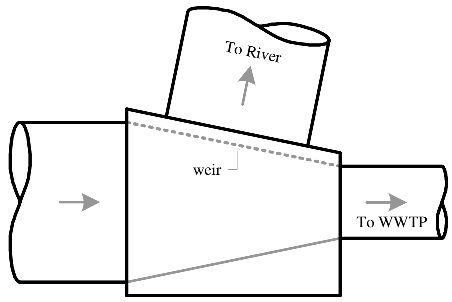


FIGURE 13: Plan View of a CSO Regulator with a Side Weir

The scattergraph shown in Figure 14 displays data from a flow monitor installed in a 3000-mm sewer located just upstream from a CSO regulator equipped with a side weir.

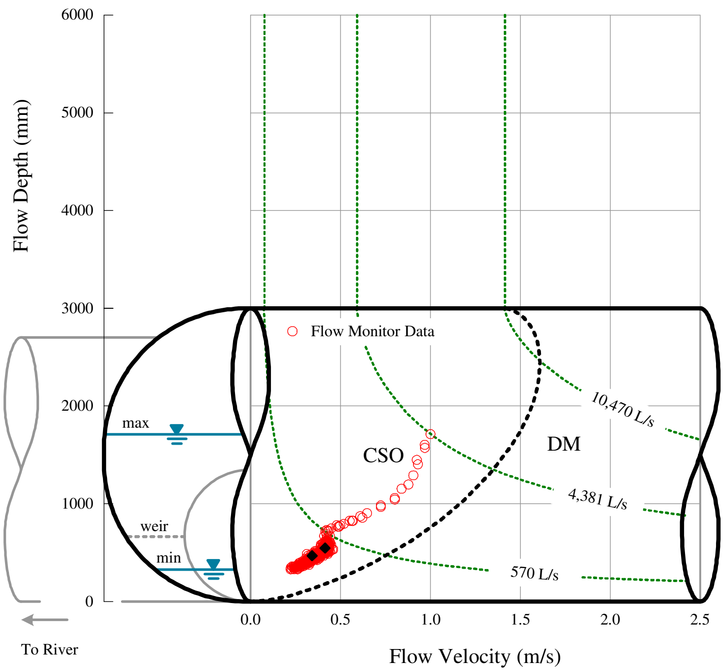


FIGURE 14: Scattergraph of a CSO from a Side Weir

Dry weather flows are funneled to the WWTP through a 1350-mm sewer. Based on the scattergraph, the 1350-mm sewer conveys flows up to 570 L/s to the WWTP, and the CSO is activated at a weir elevation of 675 mm. The maximum overflow rate is determined using iso-Q lines and is approximately 3811 L/s (4381 L/s – 570 L/s).

Overflow Duration and Volume

Once the signature of an SSO or CSO has been identified on a scattergraph and the flow depth (d_0) and flow rate (Q_0) at the onset of the overflow have been determined, the overflow duration and volume can be estimated from flow monitor data. Procedures for estimating these parameters are described in the following sections.

Overflow Duration

The overflow duration (t_{OF}) is calculated by determining the number of recorded flow monitor readings where $d > d_0$ and multiplying by the sample period between readings, as shown in Equation (1).

$$t_{OF} = n \times T \tag{1}$$

where: t_{OF} = overflow duration, s
n = number of flow monitor readings where $d > d_0$
T = sample period, s

Overflow Volume

The overflow volume (V_{OF}) is calculated using Equations (2) through (4). These three equations can also be algebraically rearranged and condensed into one equation as shown in Equation (5).

$$Q_{OFi} = Q_i - Q_0 \tag{2}$$

$$V_{OFi} = Q_{OFi} T \tag{3}$$

$$V_{OF} = \sum_{i=1}^{n} V_{OFi} \tag{4}$$

$$V_{OF} = \sum_{i=1}^{n} \left[(Q_i - Q_0) \times T \right] \tag{5}$$

where: Q_{OFi} = overflow rate at time $t = i$, L/s
Q_i = flow rate at time $t = i$, L/s
Q_0 = flow rate at onset of overflow, L/s
V_{OFi} = overflow volume at time $t = i$, L
T = sample period, s
V_{OF} = total overflow volume, L
n = number of flow monitor readings where $d > d_0$

An example is provided to demonstrate these procedures.

EXAMPLE

Flow monitor data from a 300-mm sewer are plotted on a scattergraph and suggest that an SSO has occurred downstream from the monitoring location.

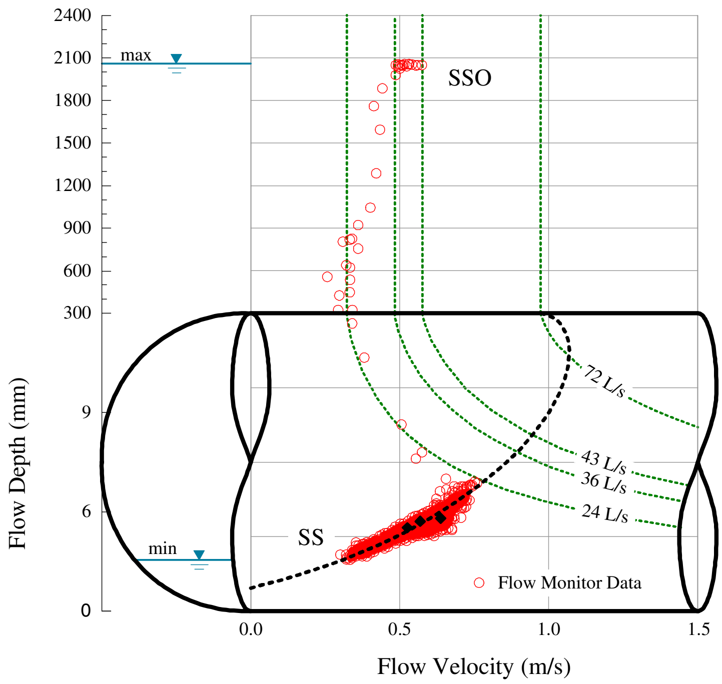
(a) Use the scattergraph provided below to determine d_0 and Q_0.

(b) Use the tabular data provided on the following page to estimate the duration and volume of the downstream SSO.

Solution

(a) A downstream SSO is characterized by an increase in flow velocity at a constant surcharge depth. Based on the scattergraph provided, this increase occurs at a flow depth of 2060 mm. Therefore, d_0 = 2060 mm.

Using the iso-Q lines provided on the scattergraph, this SSO begins when the sewer flow rate increases to 36 L/s. Therefore, Q_0 = 36 L/s.



(b) Based on the flow depth (d) data provided, the SSO begins on 03/20 at 18:00 when the flow depth rises above d_0 (2060 mm). The flow depth remains above d_0 until 03/21 at 02:00 — an overflow duration of about 8 hours.

Based on the flow rate (Q) data provided, calculate the overflow rate (Q_{OFi}) for each flow monitor reading using Equation 2, and determine the overflow volume (V_{OFi}) using Equation 3. Add the overflow volumes for each flow monitor reading to determine the total overflow volume (V_{OF}) using Equation 4. The overflow volume for this SSO is about 73,300 L.

EXAMPLE

date	time	d	v	Q	Q_{OF}	V_{OF}
mm/dd	hh:mm	mm	m/s	L/s	L/s	L
03/20	17:00	324	0.30	21.80	0.00	0
03/20	17:15	840	0.35	25.35	0.00	0
03/20	17:30	1785	0.42	30.69	0.00	0
03/20	17:45	2055	0.51	37.14	0.00	0
03/20	18:00	2077	0.49	36.03	0.03	26
03/20	18:15	2083	0.49	36.03	0.03	26
03/20	18:30	2083	0.49	36.03	0.03	26
03/20	18:45	2084	0.50	36.25	0.25	226
03/20	19:00	2084	0.50	36.47	0.47	426
03/20	19:15	2082	0.50	36.70	0.70	626
03/20	19:30	2081	0.50	36.70	0.70	1626
03/20	19:45	2077	0.57	41.81	5.81	5230
03/20	20:00	2069	0.53	38.92	2.92	2628
03/20	20:15	2065	0.50	36.70	0.70	626
03/20	20:30	2054	0.51	37.36	1.36	1227
03/20	20:45	2088	0.51	37.14	1.14	1027
03/20	21:00	2087	0.51	36.92	0.92	827
03/20	21:15	2088	0.50	36.70	0.70	626
03/20	21:30	2088	0.50	36.70	0.70	626
03/20	21:45	2087	0.54	39.59	3.59	3228
03/20	22:00	2086	0.50	36.70	0.70	626
03/20	22:15	2087	0.52	38.25	2.25	2028
03/20	22:30	2087	0.54	39.59	3.59	3228
03/20	22:45	2086	0.55	40.48	4.48	4029
03/20	23:00	2084	0.49	36.03	0.03	26
03/20	23:15	2082	0.52	38.25	2.25	2028
03/20	23:30	2083	0.54	39.59	3.59	3228
03/20	23:45	2085	0.57	41.81	5.81	5230
03/21	00:00	2086	0.56	41.14	5.14	4630
03/21	00:15	2085	0.57	41.81	5.81	5230
03/21	00:30	2082	0.59	42.70	6.70	6031
03/21	00:45	2082	0.57	41.81	5.81	5230
03/21	01:00	2080	0.57	41.81	5.81	5230
03/21	01:15	2078	0.57	41.81	5.81	5230
03/21	01:30	2075	0.52	38.25	2.25	2028
03/21	01:45	2071	0.51	37.36	1.36	1227
03/21	02:00	2011	0.49	36.03	0.00	0000
03/21	02:15	1912	0.45	32.92	0.00	0000
03/21	02:30	1617	0.44	32.25	0.00	0000
						73,287

SSO begins when the flow depth rises above d_0 (2060 mm)

Flow monitor data is obtained at a sample period (T) equal to 15 minutes (900 s)

Peak overflow rate (Q_{OF}) is 6.70 L/s

SSO ends when the flow depth drops below d_0 (2060 mm)

Overflow duration is about 8 hours

Overflow volume is about 73,300 L

Limitations

The calculation procedures described here are best applied to SSOs and CSOs that are located downstream from a flow monitor when backwater conditions are experienced prior to an overflow event. The overflow duration can also be estimated for SSOs that occur upstream from a flow monitor; however, the overflow volume cannot be determined using these procedures.

Overflow duration and volume estimates obtained using this method are a function of the quality and repeatability of the flow monitor data on which they are based. If data are missing during portions of an overflow event, the application of this method is compromised. The sample rate at which the data are obtained is also a factor. The accuracy of overflow duration and volume estimates increases as the sample rate of the flow monitor increases.

Conclusion

Sewer overflows are readily identified by evaluating flow monitoring data on a scattergraph. Scattergraph signatures of various sewer overflows have been presented in this paper, along with techniques to estimate their duration and volume from flow monitor data. These methods provide important pieces of information needed to assess the impact of sewer overflows and minimize their future occurrence.

Symbols and Notation

The following symbols and notation are used in this paper:

VARIABLES

d = flow depth, mm
v = flow velocity, m/s
Q = flow rate, L/s
d_0 = flow depth at onset of overflow, mm
Q_0 = flow rate at onset of overflow, L/s
Q_{OFi} = overflow rate at time t = i, L/s
V_{OFi} = overflow volume at time t = i, L
V_{OF} = overflow volume, L
T = sample period, s
n = number of flow monitor readings where $d > d_0$

Acknowledgement

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References

Enfinger, K.L. and Keefe, P.N. (2004). “Scattergraph Principles and Practice – Building a Better View of Flow Monitoring Data,” KY-TN Water Environment Association Water Professionals Conference; Nashville, TN.

Enfinger, K.L. and Kimbrough, H.R. (2004). “Scattergraph Principles and Practice – A Comparison of Various Applications of the Manning Equation,” *Proceedings of the Pipeline Division Specialty Conference*; San Diego, CA; American Society of Civil Engineers: Reston, VA.

Enfinger, K.L. and Stevens, P.L. (2006). “Scattergraph Principles and Practice – Tools and Techniques to Evaluate Sewer Capacity,” *Proceedings of the Pipeline Division Specialty Conference*; Chicago, IL; American Society of Civil Engineers: Reston, VA.

EPA CSO/SSO Report to Congress (2004). *Impacts and Control of CSOs and SSOs*, EPA/833/R-04/001; Washington, D.C.

Stevens, P.L. and Sands, H.M. (1995). “Sanitary Sewer Overflows Leave Telltale Signs in Depth-Velocity Scattergraphs,” *Seminar Publication – National Conference on Sanitary Sewer Overflows*; EPA/625/R-96/007; Washington, D.C.

Development of an Asset Management Framework For Culvert Inventory and Inspection

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Abstract

The deteriorating highway culvert infrastructure has become a major challenge for the 21st century. While more importance usually is given to highway embankments, pavements, and bridges, the maintenance of culverts has commonly been neglected. As many culverts reach the end of their design life, the state Departments of Transportation (DOTs) are in need of a model to track the existing culverts and forecast their conditions. Therefore, the main goals of this research is to develop a framework for culvert inventory and inspection by providing protocols and condition rating systems for culvert inventory and inspection, and validate the developed framework by conducting field pilot studies. Performance scores for the culverts are calculated using an analytical hierarchy process (AHP) to determine the magnitude of the deterioration and assist in short and long term planning. This study focuses on concrete, corrugated metal, and plastic culverts spanning less than or equal to 10 feet (3 meters). The developed model contributes to an effective culvert asset management strategy.

Introduction

Managing infrastructure is a challenging task, which requires establishment of the potential degradation of an asset over its useful life and an analysis of the impact of asset failure. Factors such as poor quality control and inadequate inspection and maintenance programs have adversely impacted underground infrastructures. The rapidly deteriorating culverts demand the local and state agencies to implement a proper inventory and inspection program. However, predicting and monitoring the condition of pipelines remains a difficult task (Najafi 2005).

Culvert inspection and management have been important topics among present day transportation researchers. The Ohio Research Institute for Transportation and the Environment, at the University of Ohio made an important contribution in their report

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"Risk Assessment and Update of Inspection Procedures for Culverts." (Mitchell et al, 2005). They introduced a culvert inspection system from data collected at sixty culvert sites. They reported that loss of culvert integrity could result in temporary roadway closure with considerable remediation costs, and total collapse of culverts could result in a major safety risk for motorists. The statistical analysis of the culverts indicated that age, rise, flow abrasiveness, pH, flow velocity, and culvert material type were significant variables for the rating system. The nationwide survey they conducted indicated that 60% of state DOTs have developed culvert inspection policies; majority specify 1-2 year inspection cycle; and a small percentage specify a 3-5 year cycle. Most of the states that inspect culverts have applied a numerical rating system.

Five states besides Ohio have developed culvert inspection manuals. Only five other states have developed their own culvert risk assessment procedures. Once the culverts are identified for remedial work in any district, the adjusted overall rating (AOR), which is the average condition rating score adjusted by the culvert age, pH of drainage water, abrasiveness of the drainage flow, and cover height to rise or diameter ratio is used to prioritize the work. The lower the AOR score, the higher the priority for repairs or replacement. None of the culverts examined had serious alignment problems. The service life of concrete culverts appeared to be limited to 70-80 years.

The most frequently encountered conditions were deteriorated headwalls, deterioration of concrete in the crown region or top slab and inlet walls, and transverse shear cracks on abutment walls. No serious alignment problems were found at metal culvert sites. No stress cracks were detected at the bolt lines inside any of the metal culverts. The service life of metal culverts appeared to be limited to 60-65 years. Also, the report suggested appropriate renewal techniques depending on structural, hydraulic, and environmental conditions of the culvert (Mitchell et al, 2005).

Culvert Inventory Data Collection Format (CIDCF)

The Culvert Inventory Data Collection Format (CIDCF) is a process of identifying and numbering culverts in a systematic and defined way. It provides a starting point for greater understanding and identification of culverts which were overlooked for years. This model is a set of useful questions in the form of protocol used to identify the culverts. The identification includes logical details of the culvert, its components, and the surrounding area. Once the culverts are identified and entered in the inventory database with a unique identification number, they can be linked to various information and decision support systems for financial, economical, and management purposes.

The model consists of fifty five questions grouped in six modules – general, structural, hydraulic, safety, repair, and additional information. All the questions should be coded as given in the inventory manual.

The general identification of the culvert location is the first module in the inventory model. This module aims in identifying the culvert from regional information to specific culvert structure. The items covered in this module are:

- *State Code*: coding of the culvert based on state codes. It follows the Federal Information Processing System (FIPS) standards.
- *County Code*: coding of the culvert based on the state counties. It follows the FIPS standards.
- *Place Code*: coding of the culvert based on the cities, townships, and villages. It follows the FIPS standards.
- *Inventory Code*: it is a unique identification number for culverts based on route signing, level of service, route number, direction, and the structure number.
- *Mile Marker*: coding of the culvert based on the nearest mile marker on the roadway.
- *Year Built*: year in which the culvert was built. This can be determined through as-built drawings review.
- *Latitude and Longitude*: latitude and longitude coordinates of the culvert can be determined using Global Positioning System (GPS) techniques.
- *Maintenance Responsibility*: the primary responsibility of the agencies in maintaining the culvert will be coded.
- *Average Daily Traffic (ADT)*: the ADT shall be determined for the route under which the culvert exists.
- *Approach Roadway Width*: the width of the roadway above the culvert shall be determined.
- *Culvert Marker*: the type of culvert marker used in identification shall be coded.

The second module in the inventory model is the structural information. This module is very important to understand the structural concepts or design of the culvert. It can be used as a benchmark to measure the structural deteriorations during inspection. The identification items in this module are: shape, material, number of cells, length, diameter or span and rise, other geometric conditions, depth of cover to the road surface, and for corrugated metal pipe (CMP) the pitch, depth and gauge.

The third module in the culvert inventory model is additional information which identifies the components of the culvert and other related features. This module acts as a benchmark for various culvert component distress or deficiencies. The identification items in this module are: end treatment type, material and thickness; slope of embankment; skew angle; roadway material; and the number of lanes.

The fourth module is identification related to hydraulics of the culvert. Hydraulic features are major factors affecting the design performance of the culverts. Identification of these features in the inventory model acts as a benchmark during culvert inspections and determines the rate of deterioration of the culvert due to hydraulic factors. The items considered in this module are: stream bed material, drainage area, peak flow, Manning's coefficient 'n', discharge, headwater depth, culvert slope, bank protection, type of fish passage, and pH of the water.

The fifth module is the identification of the safety features of the culvert like culvert grates and guard rails. The identification and assessment of these features are a part of highway safety for travelers. Also, defects in these components may indicate problems in the culvert underneath them. The items in this module are: type of safety component, material, and span.

The sixth and final module in the culvert inventory model is the identification of previous repair or renewal to the culvert. This information gives an understanding of the problems or defects existed in the culverts and methods or techniques used in repairing or rehabilitating them. The items in this module include type and date of renewal.

Condition Assessment Framework

The condition assessment framework is a set of protocols that identifies the culverts which are underperforming, determines the reasons for their deficiencies, predicts when failure is likely to occur, and develops short and long term plans for their preservation. This model is based on the inventory data collection format, literature review, field studies, and discussions with the DOTs. The condition assessment framework is divided into two categories as Basic Condition Assessment and Advanced Condition Assessment.

Basic Condition Assessment

The basic condition assessment is the general inspection of the culvert, its components, and surrounding area. It is the quickest way of collecting relevant and good information during inspection. The assessment begins with recording the general identification of the inspection site and culvert structure. Then the various components of the culvert are inspected for defects against a condition rating system. The culvert and its components are assigned a condition rating value ranging from 5 to 1; 5 being excellent and 1 being failure/critical. Using an AHP, relative weights for these components are assigned and a culvert performance score is calculated. Based on the performance score, the culvert is categorized into three zones: Red Zone which indicates verge of failure, Yellow Zone indicating intermediate stage and Green Zone indicating a safe condition.

The culverts which fall in the red zone are investigated further for "Advanced Condition Assessment or ACA." Based on the zoning, short and long term planning for culvert preservation and maintenance is implemented.

Module one of the basic condition assessments is about general information. The identification of the inspection site is necessary for any condition assessment system. The items in this module are the same as considered in module one of the inventory model. They are as follows: state code, county code, place code, culvert identification number, year built, date of inspection, inspector's name, and maintenance responsibility.

Module two is the culvert site information. Documentation of the site information is necessary because deteriorated site conditions indicate the deterioration of the culvert and its components. Also, recording the time, season, and temperature during the inspection is important because they have some influence on the effectiveness of the inspection. The items in this module are: inspection season, climate, time of inspection, type of stream, type of inspection, water level, pH of water, soil resistivity, vegetation, and natural hazards.

Module three is the identification of the culvert. Basic structural understanding is very necessary before inspection of the culverts. Comparison of the inspected geometric dimensions with the design dimensions would indicate various structural defects. The items in this module are: shape, material, number of cells, type of end treatment, and geometric dimensions.

Module four is the condition assessment of the culvert. Government Accounting Standard Board No. 34 (GASB 34) requires that a measurement or rating scale be used for condition assessment of any asset and a minimum acceptable condition be established as a benchmark. This module lists the various components of the culvert to be inspected against a condition rating system as shown in Table 1, which defines the various degree or magnitude of the defects. The inspector should carefully inspect the culvert and assign a rating for each culvert component. The condition rating system in module four for various components of the culvert is as follows:

A. Condition of the Inverts

Condition of the inverts has a major impact on the performance of the culvert. Common problems with the inverts are abrasion, corrosion, and settlement of debris. Age deterioration was seen in most of the culverts during initial field study.

Table 1: Condition Rating System for Inverts

Rating	Condition
5	Looks new or in excellent condition
4	Age deterioration is minor, no deformations of the openings
3	Age deterioration is moderate, some deformations of the opening
2	Age deterioration is significant or failure of the invert is imminent
1	Ends totally/partially broken

B. Condition of End Protection (Headwall, Wingwall)

End protections like headwall and wingwall are usually concrete structures. They should be inspected for common concrete problems like cracks, spalling, scaling, leakage, efflorescence, and reinforcing steel corrosion.

C. Condition of the Roadway

The condition of the roadway above the culvert indicates the structural or hydraulic problems in the culvert. Settlement of the roadway is a common problem and is due to poorly compacted embankment material and/or joint failure. Cracks and pavement patches indicate structural problems associated with the culvert. The condition rating system for condition of roadway is shown in Table 3.

D. Condition of the Embankment

Deterioration of embankments indicates defects in the culvert. Erosion is a common problem, which can be due to undercutting and rotation of culvert footings or severe differential settlement. Embankment defects sometimes lead to cracking in headwalls or wingwalls. The condition rating system for condition of embankment is shown in Table 3.

Table 2: Condition Rating System for End protection

Rating	Condition
5	Looks new or in excellent condition
4	Good condition, light scaling, hairline cracking, no leakage
3	Horizontal and diagonal cracking with or without efflorescence, minor rusting, leakage and erosion, minor scaling, differential or rotational settlement
2	Cracking with white efflorescence, major cracks, failure is imminent
1	Total/partial collapse of end protection

E. Condition of the Footings

Footings should be inspected for settlement along the length of the footing which is generally due to erosion. CMPs can tolerate some differential settlement but will be damaged due to excessive settlement. The stretching or compression of CMPs results in cracking or crushing across the footing. Deterioration in concrete footings may lead to distortions. The condition rating system for culvert footings is shown in Table 3.

Table 3: Condition Rating System for Condition of Roadway, Embankment and Footings

Rating	Roadway	Embankment	Footing
5	Looks new and in excellent condition	Soil in very good condition, no erosion found in and around the structure	Footing intact and in good condition
4	Minor settlement of the roadway, no cracks	Minor erosion away from the structure, no problem to the culvert	Minor erosion or cracking or settlement in the footing
3	Minor settlement of the roadway and minor cracks	Moderate erosion near the structure, no cracks on the headwall	Moderate cracking or differential settlement of the footing

Rating	Roadway	Embankment	Footing
2	Heavy settlement of the roadway or major cracks	Slope stability problem near the culvert, extensive hairline cracks found near the headwall	Severe differential settlement has caused distortions in the culvert
1	Roadway collapse is imminent	Embankment has collapsed or failure is imminent	Culvert has collapsed or failure is imminent

F. Overall Condition of Culvert

The overall condition of the culvert is determined by taking into account all the hydraulic, structural, environmental, and social factors. The analysis is done irrespective of the culvert type and size.

Module Five

Module five is the calculation of the performance score for the culvert. The steps followed in calculating the relative weights for all the components selected above are as follows:

Step 1: Each culvert component selected above is pair-wise compared with the remaining components in its importance on the overall performance of the culvert. The following table is used for pair-wise comparison.

Table 4: Condition Rating System for Overall Condition of the Culvert

Rating	Condition
5	Newly installed or lined culvert
4	Looks new with possible discoloration of the surface
3	Medium rust or scale, pinholes throughout the pipe material
2	Heavy rust or scale, major cracks with spalling, exposed surface of the reinforcing steel
1	Culvert is structurally or hydraulically incapable to function

Table 5: Scale for Relative Importance for Pair-Wise Comparison

Importance Level	Description
1	Equal Importance
3	Moderate Importance
3,4,5	Intermediate Importance
6	Strong Importance
7	Extreme Importance

Step 2: The matrix of comparison is developed after all pair-wise comparisons are made. The values entered in the matrix of comparison are based on the researchers' knowledge in culvert inspection and maintenance.

Table 6: Matrix of Comparison for Culvert Performance Calculation

	Culvert	*Invert*	*End Treat*	*Footing*	*Roadway*	*Embankment*
Culvert	1	3	3	3	5	5
Inverts	0.333	1	2	2	4	4
End Treat	0.333	0.5	1	2	2	2
Footing	0.333	0.5	0.5	1	4	2
Roadway	0.2	0.25	0.5	0.25	1	1
Embankment	0.2	0.25	0.5	0.5	1	1
Total	2.4	5.5	7.5	8.75	17	15

Step 3: The values below the diagonal are the reciprocals of the corresponding elements above the main diagonal. The next step is to normalize the column by summing all the elements in a column and dividing each element in that column by this sum. For the first column, each element will be divided by (1 + 0.333 + 0.333 + 0.333 + 0.2 + 0.2) = 2.4. Thus, new values in the first column are (1/2.4) = 0.4167, (0.333/2.4) = 0.1388, (0.333/2.4) = 0.1388, (0.333/2.4) = 0.1388, (0.2/2.4) = 0.0833, (0.2/2.4) = 0.0833. Table 7 presents the normalized matrix.

Step 4: The final step is to add all elements in a row of the normalized matrix and divide it by the number of elements in that row. So, for the first row, the new value is (0.4167 + 0.5454 + 0.4000 + 0.3428 + 0.2941 + 0.3333) / 6 = 0.3887. Similar calculations are done to obtain relative weights of the remaining rows. The relative weights of the components according to their importance level in performance calculation are shown in Table 8.

Table 7: Normalized Matrix for Culvert Performance Calculation

	Culvert	*Invert*	*End Treat*	*Footing*	*Roadway*	*Embankment*
Culverts	0.4167	0.5454	0.4000	0.3428	0.2941	0.3333
Inverts	0.1388	0.1818	0.2666	0.2285	0.2352	0.2666
End Treat	0.1388	0.0909	0.1333	0.2285	0.1176	0.1333
Footing	0.1388	0.0909	0.0666	0.1142	0.2352	0.1333
Roadway	0.0833	0.0454	0.0666	0.0285	0.0588	0.0666
Embankment	0.0833	0.0454	0.0666	0.0571	0.0588	0.0666

Table 8: Relative Weights of the Culvert Components

Type	Relative Weights
Overall Culvert Condition	0.38871
Condition of Inverts	0.21958
Condition of End Treat	0.14040
Condition of Footings	0.12983
Condition of Roadway	0.05820
Condition of Embankment	0.06297

Module six is the zoning of the culverts based on their performance. The formula for calculating the performance score of the culvert is as follows:

Performance Score of the culvert = $\sum$ Condition Rating x Relative Weight

The performance score of a culvert is a factor used as a benchmark to develop short and long term planning. Based on the performance score, the culvert is zoned into three categories – Red, Yellow, and Green. The maximum score a culvert can obtain is 5.0 and minimum is 0. A performance score higher than 3.5 indicates a green zone or safe , between 2.5 and 3.5 indicates a yellow zone or intermediate and lower than 2.5 indicates a red zone - danger.

Advanced Condition Assessment (ACA)

ACA is a detailed inspection of the culvert structure. Any culvert with a performance score below 2.5 is inspected for specific problems which have caused deterioration. The objective of ACA is to have a condition rating system for problems causing deterioration specific to concrete, metal, and plastic culverts. The assessment begins with the detailed inspection at the inlet, outlet, and inside the culvert pipe. The condition rating system between 5-0 is used as a benchmark in identifying the problems. Using AHP as described in basic condition assessment, relative importance weights are calculated for all the culvert problems as shown in Tables 9 and 10. The performance score is calculated using the formula:

ACA Performance Score = $\sum$ ACA Condition Rating x Relative Weight

The inspector needs to recommend repair or renewal to fix the specific problem causing culvert deterioration. After the culvert is treated, the performance score is calculated to check the percent performance improvement in the culvert using the formula given below:

$$\text{Percent Performance Improvement} = \frac{(PS)_F - (PS)_I}{(PS)_F} X100$$

Where, $(PS)_F$ = Performance score after the culvert is repaired or rehabilitated

$(PS)_I$ = Performance score when the culvert problem was identified or before repair or renewal of the culvert.

Table 9: ACA Condition Rating Factors and their Relative Weight for Concrete Culverts

Condition Rating Factors	Relative Weight
Cracking	0.3170
Scouring	0.1703
Settlement	0.1563
Joint Opening	0.1521
Misalignment	0.1348
Concrete Surface	0.0690

Table 10: ACA Condition Rating Factors and their Relative Weight for CMP Culverts

Condition Rating Factors	Relative Weight
Misalignment	0.2351
Settlement	0.1378
Vegetation	0.1378
Seam	0.1748
Shape	0.1748
Corrosion	0.1048
Scouring	0.0693

Conclusions

The main goal of this research was to develop a framework for culvert inventory and inspection as a part of the asset management strategy. The asset inventory and inspection model is the foundation in developing any management strategy for preserving our deteriorating culvert infrastructure. This research developed a model for culvert inventory and inspection and validated the model by conducting a pilot field study.

The culverts throughout the nation are facing significant performance challenges as many are nearing the end of their design life. These culverts are in need of special attention in terms of proactive or preventive asset management. Many DOTs throughout the nation do not have appropriate protocols to track and inspect these deteriorating culverts. The models developed in this research project for a condition rating system and performance calculator will assist the state and local agencies in making proper decisions and implementing a good asset management program.

The field study conducted in Michigan successfully identified six culverts using a unique identification number and assessed the condition of the culverts. Three out of six culverts inspected were in the danger zone. The common problems identified were misalignment, joint failure, cracking, spalling, corrosion, scouring, age deterioration, erosion of the embankments, potholes or cracks on roadway, heavy vegetation and settlement problems. The DOTs can use this model in categorizing the culverts in different zones and develop short and long term plans for each zone.

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References

Federal Highway Administration. (2001). "Culvert Management System, User Manual," *Report No. FHWA – LT – 02 – 001*, Washington, D.C.

Government Accounting Standards Series (GASS) (1999). "Statement No. 34 of the Governmental Accounting Standards Board," No. 171-A, June 1999.

Najafi, M. (2005). *Trenchless Technology – Pipeline and Utility Design, Construction and Renewal,* McGraw-Hill, New York.

Mitchell, G. F., Masada, T., Sargand, S. M., Tarawneh, B., Stewart, K. E., Mapel, S., and Roberts, J. (2005). "Risk Assessment and Update of Inspection Procedures for Culverts," *Report No. FHWA/OH-2005/002*

Perrin, J. Jr. and Jhaveri, S. C. (2004). "The Economic Costs of Culvert Failure," *Proc. Transportation Research Board 2004 Annual Meeting CD-ROM.*

Perrin, J. Jr. and Dwivedi, R. (2006). "Need for Culvert Asset Management," *Transportation Research Record: Journal of the Transportation Research Board,* Issue 1957, 8-15

Making Remaining Life Predictions for Better Pipeline Asset Management

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Abstract

Most underground assets owners would like to know what the probability is for failure of a given pipeline asset as a function of material type, function type, age of the asset, geotechnical environment, and other factors, when we know past failure distributions, predominant failure mechanisms, and other attributes. While most underground utilities have collected voluminous data that could guide them into better buried asset management in the future, the use of suitable reliability analyses have been beyond their reach. Often the replacement and rehabilitation decisions have been based on simple rules of thumb rather than either good science or statistical analyses even when tremendous amount of resources and time are expended on benefiting from the use of state-of-the-art condition assessment techniques. When civil engineers struggle to convince the public and the legislators the dire need for increased rate of investments into buried assets, it is our obligation to engage the most suitable analytical tools to make the best use of past failure data and available infrastructure capex funds. This paper provides a methodology on how sound reliability analysis tools can be used in such management decisions to maintain and operate our underground assets better.

Introduction

The United States has many trillions of dollars into its valuable underground infrastructure over the past century. The route lengths of many of these utilities are presented in Table 1. In addition, the country has electrical cables for power transmission and distribution of a route length of over 1,000,000 km and optical fiber cables of a route length of over 800,000 km. We are continuing to spend large budgets on in-situ condition assessment of existing underground assets and on forensic examination. Often, component materials forming these underground assets are also tested resulting in enormous funds being spent for calibration of data collected from other testing techniques, yet little attention has been paid on using proper statistical analyses of all of this data. Most industries outside of civil engineering have progressed much farther in the use of more advanced data analyses over the past 50 years.

Table 1. Route Lengths of America's Underground Assets

Sanitary sewers	1,280,000
Storm drains	720,000
Combined sewers	160,000
Potable waterlines	1,360,000
Natural gas lines	1,800,000
Petroleum pipelines	480,000
Irrigation pipelines	320,000
Industrial waste lines	880,000
Total	7,000,000 km

The United States spends over $10 billion in repairing and replacing leaking underground pipelines while spending over $30 billion each year in designing and building new lines. The lengths of sewers in various sizes and the portions of the pipe that have reached an age of 60 years or older are shown in Table 2.

Table 2. Size and Age Distribution of Sewers

Size (mm)	% of total	% > 60 yrs
200-300	50	5.0
375-600	21	2.1
675-1200	16	1.6
1350-1800	8	0.8
>1800	5	0.5

Similarly, the distribution of water pipes is presented in Table 3.

Table 3. Age and Material Distribution for Water Pipe

27 %	< 20 yrs old
36 %	> 20 yrs BUT < or = 40 yrs old
20 %	> 40 yrs BUT < or = 60 yrs old
15 %	> 60 yrs BUT < or = 90 yrs old
2 %	> 90 yrs old

95 % of cast and ductile iron

The questions often asked by the utilities trying to meet the requirements within GASB34 are:

What is the probability of failure of a given linear asset as a function of certain attributes such as

- type of material?
- type of function?
- age distribution of the asset?
- type of environment?
- break history?
- predominant failure mechanisms?

How do we allocate future funding to get the most optimum return from the current assets?

Take for example the estimation of remaining life of either cast iron or ductile iron pipe buried in the ground to transport either water or sewage. When one turns to the ductile iron pipe industry for guidance on remaining life of their pipe, the staff from DIPRA and their member companies would refer either the engineer or the financial advisor doing the GASB34 analyses to the DIPRA website which displays 611 utilities having cast iron pipe for longer than 100 years and 21 utilities with pipe for longer than 150 years among their underground assets. However, DIPRA's recent design decision model assumes only 50 to 75 years as the design life of ductile iron pipe, which is considered to be an improvement over the cast iron pipe that was discontinued in the 1960's. It is also interesting to note that the wall thickness of 914 mm (36 inch) cast or ductile iron pipe for example has dropped from 40 mm (1.58 inch) in 1908 to 5.3mm (0.21 inch) in 2006. Therefore, either the engineer or their financial advisor doing a GASB34 evaluation on the remaining value of the underground assets is somewhat lost on how to complete this task.

Steps in Reliability Analyses

It is not possible to rely only on the analytical tools known to engineers who have practiced design engineering, condition assessment, and asset management for water or sewer pipelines to complete the remaining life predictions. One has to use tools from other industries in such reliability studies. The appropriate steps in proper reliability analyses toward remaining life prediction shall contain:

- Collect and organize track record data.
- Select a statistical distribution that best fits the lifetime data on hand.
- Estimate the defining parameters that fit the statistical distribution chosen to represent the lifetime data for example using regression studies.
- Make better predictions than rules of thumb on estimates of the life's attributes:

  - reliability or representative life of the asset?
  - probability of failure for a chosen life span?
  - which underground component material lasts how long?

➢under what site and operating conditions?

Weibull and Normal Probability Distributions:

The Weibull probability density functions (PDFs) can be used to characterize past failure records of pipelines, if sufficient data indicate that one or both of these PDFs would approximate the past failure behavior of the buried assets.

The 3-Parameter Weibull PDF is represented by the following equation:

$$f(t) = \frac{\beta}{\eta}\left(\frac{t-\gamma}{\eta}\right)^{\beta-1} e^{-\left(\frac{t-\gamma}{\eta}\right)^{\beta}} \tag{1}$$

Where β is shape parameter
η is scale parameter
γ is location parameter
t is time
f (t) is PDF.

The cumulative distribution function (CDF), F(t), or unreliability function and the reliability function, R(t) can be obtained from f(t) as follows:

$$F(t) = \int_0^t f(t)\,dt, \text{ and} \tag{2}$$

$$R(t) = 1 - F(t) \tag{3}$$

Weibull failure rate function is given by

$$\lambda(t) = f(t)/R(t) \tag{4}$$

Some observations can be made based on the value of β. For example:

- If $0 < \beta < 1$, there is infant mortality due to either the pipes that were installed had defects at the factory, mishandled by the contractor, or the installation and inspection were poor.

- If $\beta = 1$, there are random failures independent of age, and the failure rate does not vary with time.

- If, $\beta > 1$, there are wear-out driven failures primarily due to aging and the rate is increasing with time.

Simpler Weibull PDFs can also be used when the past failure data warrant. The 2-Parameter Weibull Distribution is recommended when the location parameter, γ is set to zero and the 1-Parameter Weibull Distribution, when the shape parameter, β is a constant. In this case, the only unknown is the scale parameter, η. Note that in the formulation of the 1-parameter Weibull PDF, we assume that the shape parameter β is known *a priori* from past experience on either identical or similar underground assets. The unknown parameters that affect the location, scale, and shape are obtained using any one or more of the following techniques:

- Probability plotting
- Rank regression on x
- Rank regression on y
- Maximum likelihood estimation

The most appropriate and even whether one needs a 3-parameter Weibull, is governed by the lifetime data set on hand and good engineering judgment from experience in conducting reliability studies over the years.

The normal probability density function can be represented by the form:

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2} \tag{5}$$

where, x is the variable, σ is the standard deviation, and μ is the arithmetic mean. Again, the unreliability function, F(x), reliability function, R(x), and the failure rate function, $\lambda(x)$ for the normal PDF can be written as follows:

$$F(x) = \int_0^x f(x)\,dx, \text{ and} \tag{6}$$

$$R(x) = 1 - F(x) \tag{7}$$

$$\lambda(x) = f(x)/R(x) \tag{8}$$

Typical Results from Reliability Analyses

The useful results from the above Reliability Analyses are as follows:

- Reliability for a chosen life: what is the likelihood that the clay pipe in a sewer district will last at least 250 years?

•Probability of failure for a chosen life: what is the likelihood that the Prestressed Concrete Cylinder Pipe (PCCP) owned by the water utility will last 12 more years?

•Mean life: what is the average life of the city's entire underground asset that has certain attributes, for example, buried in CL under min 3.6 m (12 ft) of cover in slopes steeper than 6 % in areas that get more than 43 inches of rain per annum with a water table < 1 m (3.28)?

•Failure rate: what is the rate at which the water authority's underground asset will fail during the next 25 years?

•Warranty time: what is the estimated years when the reliability of the sewers would either match or exceed city council's goal driven by the council's budget constraints?

The following additional results could be obtained from the previous reliability analyses:

•Plot of probability of failure over time

•Plot of reliability over time

•Plot of probability density distribution

•Plot of failure rate with time

•Confidence levels to go with the above predictions

Three Case Histories

<u>Case History 1:</u> Using Weibull Reliability Analyses

•City with a population of over 1,000,000.

•The author did a comprehensive assessment of all three transmission pipelines bringing 100% of treated water into the city: structural, geotechnical, hydraulic, seismic, corrosion, and was able to squeeze more out of these to delay capex on a 4^{th} pipeline.

•3,360 km(2,100 miles) of pipe form their distribution system assets with 6,200 breaks over 1977-2002 with pipes going back to 1890s as shown in Table 4.

•They asked for guidance to develop a better asset management system.

•Toward better allocation of their limited funds.

• The results of the 3-parameter Weibull data fit is shown in Fig. 1.

•The results of the Weibull reliability analyses on remaining life for the cast iron and galvanized pipes are shown in Fig. 2.

<u>Case History 2:</u> Using Normal Probability Density Functions

The author was asked to review the data collected, perform an analysis, and make recommendations for an asset management program after the field data have been collected without his input. Unfortunately, the condition assessment program was not properly designed and the technologies used were not the most suitable. The data collected did not capture all of the past failure patterns. The following summarizes the situation:

•Results of NDT on ductile iron wall thickness along the alignment varied
•Depth of cover varied
•Live load varied
•Internal pressure varied
•Trench condition varied

Each of these were represented by a normal PDF and factors of safety for 66 %, 90%, and 99% confidence levels were predicted to meet AWWA C-150 standards for

•external load induced deflection
•external load induced bending stress
•internal pressure induced hoop tension

to determine which portions of the alignment need to be replaced or relined and the timeline.

<u>Case History 3:</u> Using Normal Probability Density Functions

•PCCP design wall thickness, coating, core, etc. varied
•Depth of cover varied
•Live load varied
•Internal pressure varied
•Level of wall thickness loss due to H2S attack varied
•Wraps of prestress wires varied

Again, proper condition assessment techniques were not used with the evaluation of the pccp present in the force main. Each of these variables were represented by normal PDFs and AWWA C-304 design checks for 66 %, 90%, and 99% confidence levels were made using an excel sheet the author developed. An asset management program based on the results used the following factors:

•Proximity to the river and the level of damage it might engender.
•amount of concrete core loss due to corrosion
•relative aggressiveness of native soils
•surge potential and the working pressure
•intensity of soil and live loads
•relative accessibility to the force main

Table 4 Sample Pipe Break Data

Year	Total Breaks	CI Breaks	Galvanized Breaks
1972	20	16	4
1973	30	24	6
1974	40	32	8
1975	50	40	10
1976	60	48	12
1977	75	60	15
1978	110	88	22
1979	110	88	22
1980	110	88	22
1981	130	104	26
1982	175	140	35
1983	275	220	55
1984	260	208	52
1985	300	240	60
1986	250	200	50
1987	290	232	58
1988	330	264	66
1989	360	288	72
1990	390	312	78
1991	310	248	62
1992	360	288	72
1993	280	224	56
1994	240	192	48
1995	280	224	56
1996	280	224	56
1997	200	160	40
1998	280	224	56
1999	210	168	42
2000	290	232	58
2001	200	160	40
2002	160	128	32

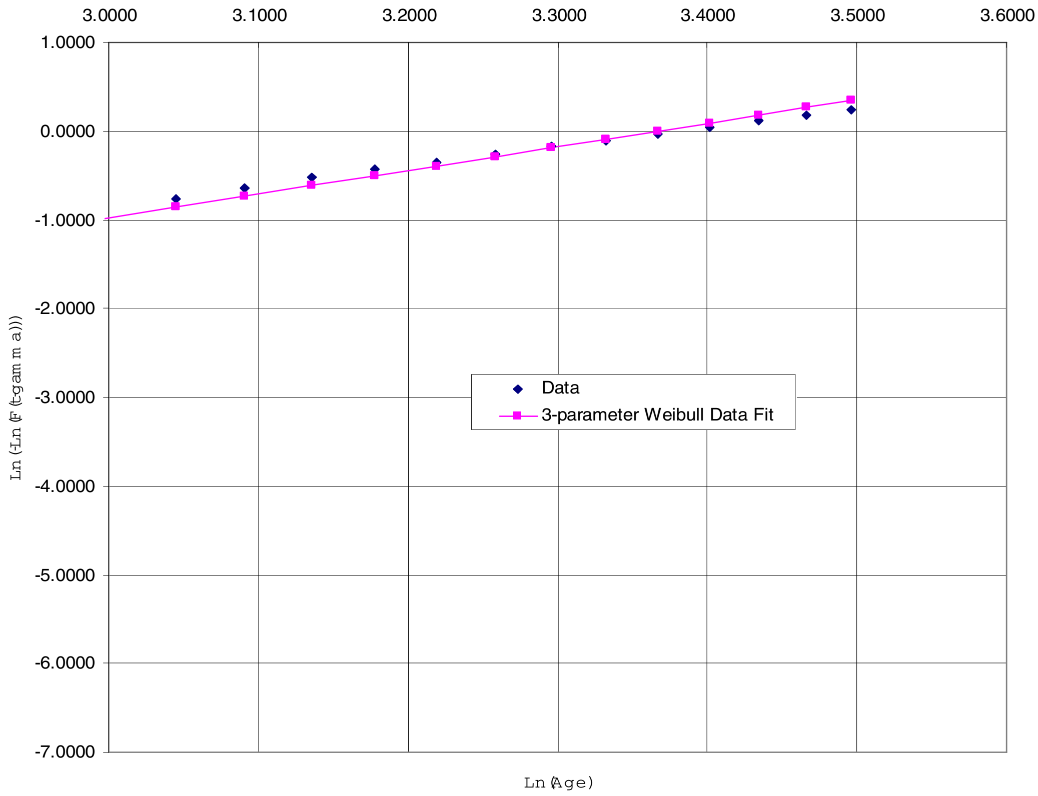

Figure 1. Data Fit for a 3-Parameter Weibull Model

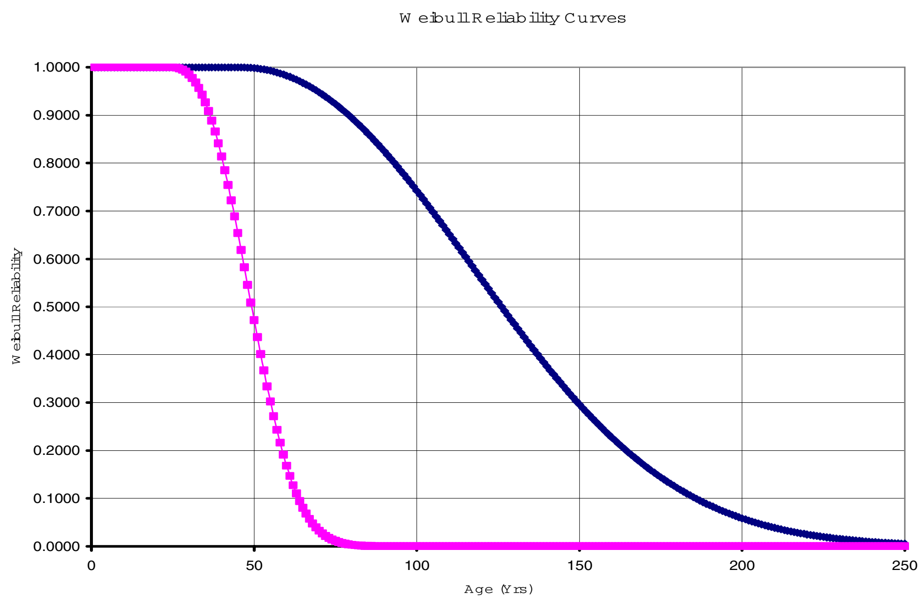

Figure 2. Weibull Reliability Analyses

Conclusions

The following conclusions can be made:

1. It is extremely important that civil engineers engage outside the box thinking to improve the delivery to our clients vis-à-vis serving our public better.

2. For example, if we were ever asked to run the oil business, we might put airlines, power plants, trucks, trains, cars, and heating, all on jet fuel and this would be so wrong. Unfortunately, this is what we have done with water for over 100 years and we have not been listening to our public on what their needs have been. Just like our people do not trust the electric utilities, a time will come when our civil engineering decisions will be questioned by the rate-paying public, if we do not change the way we do our business.

3. It's time we either use dual water lines or supply brown water for point of use treatment. Design and construction of massive centralized water treatment plants should be reconsidered, given that only 3% of the water we treat is for human consumption and the same water from the tap is sold in bottles at a profit of over 7,000% to become a highly profitable bottled water industry exceeding $ 25 billion in 2007 with 12 % year to year growth.

4. Shift happens and the owners of these projects are shifting more and more of their budgets toward design, construction, operation, and maintenance of the conveyance part of our water and wastewater systems, while cutting the budgets for the treatment side of the business.

5. The engineering tools we use for condition assessment and asset management also need to account for past failure records, variability in material properties, construction practices, loads, O&M, site characteristics, etc.

6. It is not possible to obtain a better outcome from our work for our clients, if we keep doing the same thing over and over again. It is absurd for licensed civil engineers to base their asset management decisions on condition and criticality factors that involve nothing more than a simple addition. Our efforts in underground pipeline condition assessment and asset management have to include more rigorous statistical evaluations of high quality data.

7. The three case histories presented in this paper using either Weibull or Normal PDFs are steps in the right direction in the use of reliability analyses in underground asset management. Analytical tools such as Markovian models, non-linear programming and dynamic programming techniques, Monte-Carlo simulations, Fuzzy sets, etc. would provide us with even more computational power in our ability to better allocate funding for future underground asset management programs.

8. When asked of Wayne Gretzky about his most important advice to younger players he answered “really simple; always skate to where the puck is likely to be.” It is not possible for us to see ahead clearly without looking back. The pursuits in our asset management work takes us back to the advice of Marcus Tellius Cicero during 106 to 43 BC: History is the witness of the times, the light of truth, the life of memory, and the witness of life.

Acknowledgements

The author considered applying the above methodology for underground asset management by reading the emails written by Mr. Terry Martin of Seattle Public Utilities to his counterpart in another city on how he was attempting to apply these techniques to manage Seattle sewers better. Dr. Sri K. Rajah of HDR was of immense help with using the statistical functions. Professor Samuel Ariaratnam of Arizona State University reviewed a draft of this paper and offered many valuable suggestions for improvement. The author is most grateful to these fine people.

Using GIS for Pipeline Data Management at the Palo Verde Nuclear Facility

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Abstract
Arizona Public Service (APS) operates the Palo Verde nuclear facility in central Arizona. Palo Verde is the nation's number one power producer with a net generating rating of 3,875 megawatts that provides electricity for up to 4 million people in the Southwest United States. Reactors at the plant depend upon pre-stressed concrete pipes to transport the coolant water throughout the system. More significantly, Palo Verde is the only nuclear plant in the world not built on a large water source such as a lake, river or ocean. More than billion gallons a year of municipal effluent is recycled each year and travels through 35 miles of piping.

Naturally, for a system this critical, a pipe failure would be disastrous, as would an extended repair outage. Engineers at the plant decided that the paper map and numerous different file systems that held the pipeline information were an inefficient manner in which to gather data and expedite decision-making. Deciding upon a GIS-enabled database, the APS engineers worked with Pressure Pipe Inspection Company (PPIC) towards an intuitive and dynamic GIS based system that provides users with timely information.

Using GIS technologies effectively for a water pipeline requires a detailed data structure to address the jointing aspects specific to water pipelines. The data model needs to track individual pipe segments, from bell to spigot, with inspection and integrity information. Enabling this database for view through GIS software delivers a GIS with a high value data set that is very accurate at representing existing water pipeline infrastructure. Implementing such a system is not a trivial procedure; however, by utilizing resources from various departments (i.e. engineering, IT, GIS and database administration) at APS, completion of the project occurred within the budget and time period allotted.

Applications within the GIS were built to enhance the casual GIS user experience. With these applications leveraging off the pipeline data model, APS pipeline operators can fully realize the advantages of maintaining their pipeline assets in a spatial database including 1) increased data availability; 2) identification of urgent repair needs; and 3) improved maintenance and capital planning.

INTRODUCTION

Arizona Public Service (APS) operates the Palo Verde nuclear facility in central Arizona. Palo Verde is the nation's number one power producer with a net generating rating of 3,875 megawatts that provides electricity for up to 40 million people in the Southwest United States. Reactors at the plant depend upon pre-stressed concrete pipes (PCCP) to transport the coolant water throughout the system. More significantly, Palo Verde is the only nuclear plant in the world not built on a large water source such as a lake, river or ocean. More than a billion gallons a year of municipal effluent is recycled each year and travels to the plant through 37 miles of piping.

Given the critical nature of this water piping, a rupture anywhere along this line would be disastrous. Extended repair outages would also compromise the ability of the power plant to supply clients with electricity. In order to take proactive action against such an incident, the plant engineers determined a need to assess the condition of this water piping. The assessment began in the 1980's with a detailed look at the design of the pipeline and the surrounding soil conditions.

In 1998, APS began its use of the Remote Field Eddy Current / Transformer Coupling (RFEC/TC) technique and found that this technique was effective in both identifying PCCP pipes with broken wires and quantifying the damage. APS has continued its use of the RFEC/TC technique, assaying a portion of their pipeline each annually. Through the inspections, APS has been able to identify and selectively rehabilitate a number of damaged pipe segments. This has further improved pipeline integrity. With at least three inspections on the entire system, this technique now provides APS with an estimated rate of deterioration. APS continues to inspect the pipeline using RFEC/TC at regular intervals in order to monitor the deterioration, if any, and refine their risk assessment modelling and rehabilitation priorities. The utility estimates that the total cost of the inspection and rehabilitation measures have extended the life of the pipeline and saved millions of dollars.

CUMBERSOME CONSTRUCTION PLANS AND FILES

The nature of a nuclear power plant necessitates current and accessible data for APS staff. Between pipeline design, manufacturing, construction and assessment information, APS possesses an abundance of information concerning the Palo Verde facility. Maintaining and accessing the data is a challenge because the numerous engineering maps, spreadsheets and binders are located throughout the facility. Furthermore, through examination of the results presented by the condition assessment program, the engineers at APS noted that their original construction plans did not accurately reflect the existing current environment around their pipelines. As APS's original construction drawings did not incorporate new land use or surface infrastructure information nor integrate condition assessment or pipeline rehabilitation data, APS's engineers were finding it challenging to model, analyze, review or record changes to their pipeline infrastructure using their current cataloguing system of paper maps and plans, CAD drawings and Excel files. APS, moreover, needed a decision facilitation tool that would help the engineers decide which pipes to repair; discover why specific pipes were corroding; and predict which pipes were going to fail.

APS had already worked with The Pressure Pipe Inspection Company (PPIC) to evaluate the condition of 37 miles of its transmission mains using the RFEC/TC condition assessment technology. During a RFEC/TC inspection, technicians convey a tool through the interior of the pipeline, sending an electromagnetic signal through the pipe wall. As can be seen in Figure 1, the tool is nominally used to pinpoint wire breaks in PCCP (irregular spikes) but is electromagnetically sensitive to the large amount of steel present at the pipe joints (regular spikes). The regular spikes in the signal identify the start and end of each pipe segment.

As part of a condition assessment report, PPIC also provides the axial location of any wire breaks discovered in a particular pipe segment and the position of these pipes as related to readily identifiable pipeline features. Due to its ability to spot joints and identify feature pipes, APS's engineers concluded that RFEC/TC provided more accurate pipe segment data than the original paper drawings. As a result, APS asked PPIC to amalgamate their diverse data sets into a Geographic Information System (GIS).

APS'S OBJECTIVES FOR A GIS

APS had the following goals in mind for their pipeline GIS:

1. Development of an accurate system map detailing each pipe section from bell to spigot;
2. Transfer all pipe section geometry, manufacturing, design, inspection, and rehabilitation information into a relational database that is GIS enabled;
3. Have the ability to use the information stored in the GIS database for developing and analyzing pipe prioritization plans.

GIS Development

A GIS is more than just a computerized map as it combines a digital map layer with a relational database. Pipeline operators must handle enormous amounts of information. This information needs to be accessible and current in order to expedite decision-making. Using a relational database is the most practical method to manage, organize and analyze this data. A relation database offers advantages such as centralized control, reduced redundancy, efficient data sharing, rules for data editing, and data independence from applications. The database and data model used with the GIS are the essence of its functionality. In effect, a GIS map is the visual rendering of information stored in a database so the better the data going into the database, the better the data coming out.

Designing the database is the first step to creating a successful GIS environment. The database design has to incorporate the needs of the users and must be flexible enough to accommodate present and future data. Developing a poorly designed database leads to a poorly designed GIS. The key to effective decision making requires combining diverse data sets into a decision facilitation matrix that can help make decisions on which pipes to repair; discover why specific pipes are corroding; and predict which pipes are going to fail.

Consequently, a water pipeline data structure must:

1. Track and manage pipeline assets effectively.
2. Provide flexibility in supporting different decision processes.
3. Provide flexibility in merging and supporting diverse data sets.
4. Provide scalability to integrate other business management tools into the system.
5. Provide the capability to examine trends in the data.

In assessing the available data structures, PPIC examined a wide array of possible options including Dynamic Segmentation, PODS and the Arc GIS Water Wastewater data model. PPIC noted that these options are suitable for collecting traditional water related infrastructure asset management data. As an example, they can incorporate fire hydrant testing results to establish C factors for waterlines. These systems are particularly suited for water supply capacity planning as hydraulic analysis software uses the same data structure, however; none of these data structures adequately addressed the pipe segment jointing aspects of water transmission mains that typically consist of short (12 ft, 16 ft & 20 ft) segments with bell and spigot ends. The node line representation model that the oil and gas industry uses, identifying features like air valves (nodes) and long reaches of pipe (line) is therefore not applicable.

With the introduction of the RFEC/TC technique in 1997, advances in the condition assessment of PCCP allow pipeline operators to make decisions on a pipe-by-pipe basis. The link node data structure could not accommodate this level of information. To model and develop applications for their PCCP pipelines, and thereby enable their engineers to integrate and track data at the individual pipe level, pipeline operators need a data structure that addresses the jointing aspects of PCCP.

Pipeline operators base their rehabilitation strategies on different criteria. Decisions are often driven by the level of distress observed in a given pipeline, the availability and accessibility of other data sets, available budgets and an individual utility's tolerance for risk. For example, some water pipeline operators choose to replace only those pipe segments that had actually failed. Others choose to replace pipe segments with a specific number of wire breaks, and all of the pipes within a 500' radius of the identified pipe.

In developing a data structure for APS, PPIC therefore took into account:

- The need to manage information from diverse data sets, enabling the development of custom criticality indexes;
- The need for a flexible system that could take into account the fact that this data changed over time, enabling the management of risk mitigation systems such as cathodic protection;
- The need to support a decision-making process whose inputs would change over time as more information became available, enabling the prioritization of selective pipe segment replacement;
- The need to support an asset inventory that is reflective of the current environment in and surrounding the pipeline enabling the development of an accurate and dynamic system map that could be used to definitively locate individual distressed pipes.

Moreover, the pipes and other system features required the system to incorporate analytical and historical tracking. Accurate failure assessment requires not only knowledge of a pipe segments condition but also the position of defects on the pipe. For APS, each defect is located axially and/or circumferentially (in the case of cracks) in the pipe segment. Storing this information in the data model allows users to track defect growth on both a pipe-by-pipe and global basis. For example, one inspection may observe two mortar cracks in a pipe segment at 1.5 meters and 2.5 meters from the downstream end. A subsequent inspection reveals one long mortar crack centered 2 meters from the downstream end. With the data model in place, a new inspector or analyst quickly knows that the two defects have become one and not the sudden appearance of a delaminated mortar coating.

APS Data Structure Implementation

As the first step in implementing this new pipe specific data model, APS and PPIC worked together to conduct a data audit – collecting all of APS's diverse range of pipeline related data sets. APS handed over its array of lay sheets, spreadsheets and notes, which PPIC organized into a logical data structure.

As the next step, accurately positioning the pipeline required precise pipe positional data and pipe segment lay information. APS already had survey grade GPS co-ordinates for a variety of surface features including Air Valves, Blow Offs and Manholes, which simplified the process. Pipe segment lay information, taken from the RFEC/TC inspection results, further uniquely identified each pipe segment between these surface features. Consequently, PPIC was able to incorporate this information into a spatial database.

After adding the pipe geometry to the database and giving each pipe segment a unique identifier, the information related to each pipe segment (e.g. design, manufacturing, inspection, rehabilitation, drawings, etc.) was populated in the database as relational tables. The database stores information for each individual pipe segment, from bell to spigot, with inspection and integrity information. Further, APS provided survey information for the land parcels located in the pipeline right-of-way. This information provides details regarding the landownership in the pipeline right-of-way. By incorporating parcel data into the pipeline database, APS can notify landowners of any work proceeding on their property.

Custom User Applications

Once data is contained in a database, writing applications to take advantage of the data structure and central storage is possible. APS pipeline operators are busy with the ongoing task of safely maintaining a nuclear facility. Adding an overly complex piece of software to their daily work life would be an inefficient and unnecessary burden.

Custom applications built within the GIS environment enhance the GIS experience for the casual GIS user and help to manage and analyze the pipeline data. These applications leverage of the data structure of the database and allow the user to:

- Identify urgent repair needs;
- Query the database to locate pipes that possess a specific attribute, or combination of attributes;
- View these attributes, as well as any other data set, documentation, image, or historical record, for an individual pipe;
- Produce graphs and charts based on any combination of fields or tables contained within the database.

The tools are simple to use and provide results in a timely manner as they access data stored in the GIS database. The system is designed to help its clients make better decisions by making use of their available data.

INTEGRATING THE GIS WITHIN APS

APS is a large organization with mature database, IT and GIS departments. PPIC cooperated with all these departments to install and integrate the new pipeline GIS for the pipeline engineers. Firstly, the IT department suggested installing the GIS software and database on a central server accessed over a Citrix network neighbourhood. A Citrix network allows a powerful central computer to do the processing work while smaller, less powerful machines provide the user interface. A user can log into the main computer from any location to observe and manipulate their GIS data. The advantages of a centralized processing computer such as this include savings on application costs, less computer downtime, and improved user support and security.

Secondly, the database administrators aided with transferring the GIS database that PPIC created into its own Oracle database for the pipeline group to use. The DBA also set up the user privilege functionality in order to make the database secure. Finally, the GIS department installed the GIS software, in this case ESRI's ArcView and the custom applications written for the pipeline group by PPIC. The APS GIS group also set up the standard look and feel of the map interface for the pipeline group as well as aiding the DBA with user privileges and transferring the GIS database to Oracle.

Using the in-house resources at APS saved the utility a considerable amount of money and at the same time allowed the staff to become familiar with the new database and software. Thus, in the future, allowing APS staff to perform any necessary updates, modifications, etc., without having to make enquires to external consultants.

CONCLUSIONS

APS's need for a system that accurately reflected the existing current environment around their pipelines, incorporated new land use and surface infrastructure information and integrated condition assessment and pipeline rehabilitation data spawned the development of a comprehensive GIS-based system for managing pipeline assets.

The goal in the development of the system was to facilitate APS's use of available information sets to allow them to make the best decisions possible. The system, therefore, allows for the accurate combination of diverse information sets so that APS can make effective use of condition assessment, environmental and trending information sets. This foundation for informed decision-making will allow APS to maximize the safe and economic life of the pipe while minimizing capital expenditures.

Reconciling Conflicting Utility Location Data: A Case Study of Submarine Gas Lines in the Providence River

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Abstract

During the course of a major harbor dredging project it was learned that three ten-inch diameter, high-pressure gas lines traversing the Federal Channel may not have been buried to the proper depths. After researching the plans of record on the subject gas line, the USACOE found that the originally permitted burial depths of the pipeline conflicted significantly with two more recent electronic surveys that had been performed. Several more surveys using electronic methods of locating the pipes were attempted; however, because of site constrictions and the pipeline configuration, functionally repeatable results could not be obtained.

Because electronic surveys had not produced consistent results, the owner and the USACOE turned to the most reliable method – that of unearthing portions of the pipe, however the Channel depths, the pipe cover and port traffic made this a formidable task. The following case study documents how this task was accomplished.

Introduction

The U.S. Corps of Engineers (USACOE) was in the process of performing improvement dredging of the Federal Channel in the Providence River to a design depth of –41' Mean Lower Low Water (MLLW), plus two feet of allowable overdepth. The channel design had taken into account the permitted burial depth of three ten-inch diameter steel gas pipelines that crossed the river. The original permits for the pipelines indicated that they had been buried to a depth of –50 feet MLLW, which would have provided adequate coverage under the DOT regulations. During the course of the dredging project it was learned that the actual records of the pipeline installation (in 1954) were fragmented – and in fact there were no realistic as-built surveys of the pipes. After considerable study, the USACOE determined that the originally permitted burial depths of the pipeline conflicted significantly with two more recent "electronic location" surveys, which were performed by the pipeline's owner. Because of the disparity between these surveys, and the fact that recent surveys showed the pipelines to be significantly shallower in depth than the permits allowed, the USACOE determined that the survey data of record was too ambiguous to be relied on, and as such it could not dredge much of the 150 foot wide corridor directly over the pipeline route. This area became known as the "no-dredge" zone and it extended 415 feet into the Federal Channel from the Western channel limit, and 305 feet into the Federal from the Eastern channel limit. Pursuant to this issue, the USACOE sent a directive to the pipeline owner to provide a functionally accurate, certified survey of the pipeline, and if the pipeline was found to be out of compliance with permits, to correct or remove same. The directive required physically locating and identifying each of the three pipes in a number of locations, surveying each pipe's position at these locations, and then having a Registered Professional Engineer certify that the surveyed locations were correct. Because of the conflicting record data, electronic and/or sonic methods of pipeline location were not considered acceptable means of survey.

The Utility owner contacted the engineering and surveying services firm (CLE Engineering, Inc.) who was already working on site for possible solutions to this directive. This engineering firm was requested to develop and conduct a survey that would satisfy the USACOE's requirements. This assignment was problematic because it required that the pipes be uncovered in a number of locations, at water depths exceeding 55 feet and because the issue occurred at the time of the winter heating season, the pipelines could not be de-pressurized for more than a short time during the investigation. This meant that the pipes had to be uncovered and measured for depth while "live" and in use which required that a number of safety related issues be addressed. Also, because the pipeline owner was concerned about the long-term life of the pipe, an excavation methodology needed to be devised that would guard the pipe and coating against any possible damage during the excavation process.

Methodology

Attempts to Locate the Pipes by Electronic Methods

The history of electronic pipe location attempts began in 1995 when the pipeline owner contracted with a diving company to perform a survey using a combination of electronic sounding and conventional hand probes. This party produced a plan and profile of the pipeline crossing, however because of a significant error in the vertical datum applied; the pipe appeared much deeper on the profile than it actually was. Since there were no plans for dredging of the river on the horizon at that time, the utility owner did not have any reason to look for a problem and the drawing was simply placed in the file. During that same time period, another company performed an electronic pigging survey of the pipe's interior, however the pig's electronics were limited and the pig only recorded pipe interior condition, length and approximate location of bends, thus it was of little help identifying the pipe's physical location. It was not until the beginning of the dredge project in 2003 that anyone had reason to look more closely at the 1995 survey.

As part of an area study relating to a concurrent dredging project, it was requested that the utility owner provide copies of any plans and/or surveys of the pipe. When these plans and the 1995 survey were reviewed it became apparent that there was an obvious datum error on the later survey. Further, when an approximated correction was applied, it placed significant portions of the pipeline within the dredging template. The USACOE was notified of the problem, and in turn they classified the suspect areas as a "no dredge zone" until the problem could be resolved. This prompted the need for follow-up surveys to attempt confirmation of the pipeline burial depth. The Utility owner re-contracted the same diving company to return and re-survey the pipeline, this time under the observation of the USACOE. The diving company employed essentially the same methods as they had used in the 1995 survey, however this time using better tidal correction data. The resultant profile was completed and when the profiles were compared, they showed essentially the same burial depths as the (corrected) original survey. The Utility owner's engineering firm was then contracted to locate a more sophisticated electronic location system to survey the pipeline. However, the search became problematic, in that the configuration of the pipeline valve pits, and several abrupt bends in the pipeline crossing prohibited the use of more accurate "tracking" pigs (due to the length of the pig). The search however did lead to one company, the specialized in Pipeline Tracking Systems, who had a technology whereby the pipes were isolated electrically, and then a "loop wire" was added across the waterway, connecting the extreme ends of the underwater crossing. Thereafter a signal generator was connected to produce an electro-magnetic field on the pipeline. The signal was then traced by running a survey boat back and forth across the pipeline at 20 to 50 foot offsets, recording bottom depth, pipeline horizontal position and pipeline burial depth. The system data was calibrated by physically probing and locating the pipe in a few known locations where the pipe burial was minimal. The survey resulted in a pipeline plan and profile that again differed considerably from earlier surveys. The most dramatic difference was found in the pipe alignment, whereby the new electronic survey showed the pipe's alignment bowed approximately 50 feet to the south of the "straight" alignment shown on both the as-built trench plan as well as the diver surveys. The new electronic survey also showed the estimated pipeline depth profile to be very close to the original trench survey (albeit somewhat different in several critical locations) and considerably different than the diver surveys of 1995 and 2004.

Nonetheless, there remained a number of unresolved disparities between the various plans and surveys. There was the (1) original Permit Plan, (2) the informal trench survey by the original installation contractor, (3) the 1995 diver survey, (4) the 2004 diver survey and (5) the 2004 Electronic survey. Even though the 2004 Electronic survey appeared to be the most accurate and reliable of all, the USACOE, in concurrence with the Utility's Engineering Firm agreed that because of the potential risk to public safety, some form of physical confirmation survey should also be performed to assure all parties of the actual pipe location. It was also determined that no dredging would be done within 75 feet of the pipeline corridor until the location was physically confirmed.

General Description

There was much concern on the part of all involved parties that any unearthing of the pipelines in the marine environment would cause damage or leaks in an area where they would be very difficult to repair. In response to this concern, a plan was developed to carefully unearth each pipeline and physically locate it with a reasonably high degree of accuracy. The methodology, proposed in the permit applications to the USACOE, Rhode Island Coastal Resources Management Council (CRMC), and the Rhode Island Department of Environmental Management (DEM) was modified slightly in the field as existing conditions dictated. In general the following procedures were followed:

The plan was to remove silt and clay materials overlaying the pipeline in selected various locations and the pipe was to be identified and then physically surveyed at these locations. Proper surveys conducted under these conditions would determine the precise vertical and horizontal position within 0.5-foot tolerance vertically and under three feet horizontally. The pipeline was to be exposed in a number of locations within the Federal Channel as well as both the East and West U.S. and RI Harborlines, dependent on field conditions such as the presence of rip rap or unstable side slopes.

The process of physically uncovering the pipes was one of the biggest challenges. There was a high degree of concern by all parties that the pipelines not be damaged in the process of uncovering them. Unfortunately, this precluded most of the common, more productive methods of marine excavation. Methods that utilized clamshell buckets or cutterhead-suction devices were ruled out. Because of the large disparity between surveys the depth of excavation could range from as little as six feet, to as much as eighteen feet below the existing harbor bottom. Water depths also made the process difficult, as the areas of the most critical verification surveys depths ranged from 34 to 40 feet, with the addition of as much as five feet of tide. It was determined that the most viable and safe method to uncover the marine sediment and clay from over the pipes was to use a marine “airlift”.

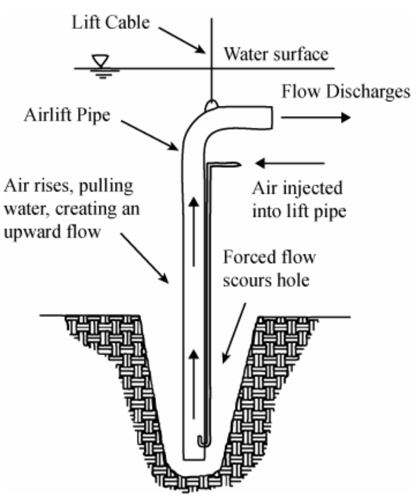

Fig. 1 Simplified diagram of how airlift works.

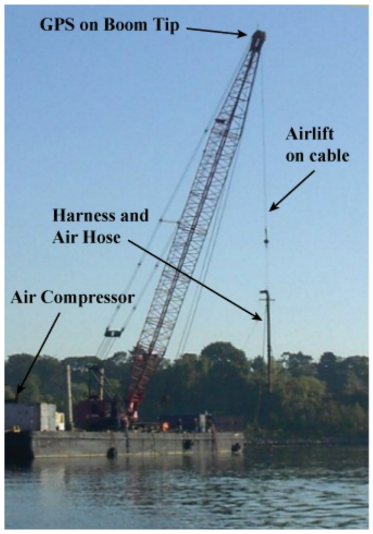

Fig. 2 Airlift suspended from cable, preparing to be lowered into water to begin excavation.

An airlift is a dredging tool that works by injecting compressed air through a small diameter pipe (2”) at the bottom of larger diameter pipe (12”). The air is injected into the bottom of the larger pipe that is positioned vertically on the river bottom by use of a cable suspended from a barge-mounted crane. The advantage of the airlift is unique in that it has the ability to scour and remove marine sediments without the need for hard digging devices, such as clamshell buckets. Instead, the air injected at the bottom of the airlift pipe rapidly rises up to the top of the pipe and the bubbles of the air stream and create a forced upward flow of water and air within the pipe. The airlift utilizes the velocity of the combined flow to agitate and liquefy marine soils, and literally suction them up the pipe and away from the area of intended excavation. Accidental abrasive damage to the pipe and coating system was prevented by the use of a rubber shoe fastened to the bottom of the airlift. Like a giant underwater vacuum cleaner, the airlift gently removed the soils overlying the pipes until they were exposed. This same technology is often used for underwater excavation of delicate archeological sites.

There were 21-targeted areas where the pipe was to be located. At each location an area of exploration was targeted; as each hole was excavated, a diver would enter the hole to attempt physical location of the pipe. Because clouds of suspended sediment remained in the excavation, diver visibility was non-existent once the diver entered the hole. From that point on all pipe location inspections were strictly done by feel. Unfortunately, the marine soils covering the pipes varied considerably in consistency, ranging from gelatinous muck to moderately stiff marine clay. This caused a large variety of excavation formations, each of which had to be individually verified for stability progressively as the diver entered the excavation. Only professional divers, with backgrounds in what are known as “black water” penetrations, with surface supplied air, emergency air and radio communication were employed.

Detailed Work Sequence

The work platform was a 40’ by 160’ barge equipped with a Model 4100 Manitowoc crane, plus two 1,300 cubic feet per minute (CFM) air compressors, with three outfitted cabins to serve as all weather enclosures for men and equipment. There were facilities for diver operations, electronic position monitoring, and equipment storage.

Barge and Airlift Positioning

The engineering firm was tasked with the responsibility of providing positioning for both the barge and airlifting operations. Both functions were critical to the investigation effort, the first system being required to properly position the barge in relation to the alignment of the pipeline, and the second system to accurately position the airlift over projected pipe locations.

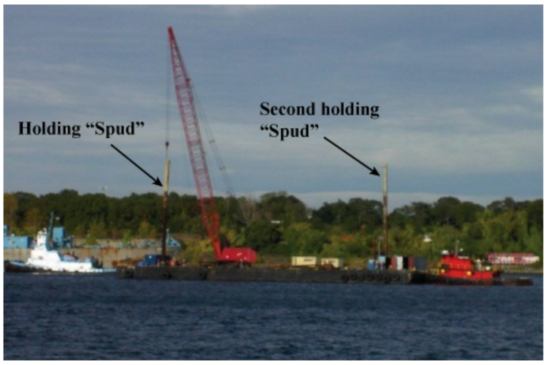

Fig. 3 Crane barge being pushed into position adjacent to pipeline.
Note: Crane is about to lower and "set" one of two holding spuds.

The method determined to be the most reliable for holding the barge in place during airlifting was the use of spuds, which consisted of two large 30" diameter pipes. Spuds were determined to have an advantage as they had far less tendency to slip or drag such as anchors might, and they confined barge movement (drifting) to a few feet while on station. The negative feature required setting the spuds near the exact limits of the designated "no-spud" zone, which was 75 feet from the anticipated center of the pipeline corridor. This was complicated because it was learned that there was not only a disparity in the pipe's depth, but there was also a significant disparity in where the pipes laid with respect to its permitted alignment. (One of the earlier electronic surveys showed the pipes bowing as much as 50 feet off line to the south of the previously designated corridor.) As such, the position of the pipeline in relation to the no-spud zone changed as the work approached the center of the river, to a point where it was within a few feet of the southerly "no spud" limit, which in turn made barge positioning extremely critical in these areas. The positioning system for the barge was a "Vector" antenna (heading accuracy to ½ degree and submeter positioning) integrated with a hydrographic graphic and coordinate display software package. This was necessary in order to identify the position of both spuds in relation to the "no-spud" zone before they were set into the bottom. The airlift positioning system was a single antenna GPS with sub-meter accuracy. The boom tip GPS also served a second function, acting as a redundant system to confirm the position of a spud before it was set. Barge and airlift positioning systems were monitored by a Field Engineer using a tracking computer with two monitors. Each showing the anticipated pipeline locations, the barge position and alignment, as well as the airlift position, all in real time. One monitor was located at the computer station, and a second monitor was located in the cab of the crane. Prior to the start of work, the entire system was calibrated against known control, including the existing pipeline valve boxes. During the progress of the project the location of each test excavation was logged and the location of the hole was recorded on the screen.

As a precautionary measure, during the initial exploratory phase of the investigation the pipeline Utility Owner opted to shut off the flow of gas through the pipelines to allow testing of the work and safety plans. Because neither the horizontal location nor depth of the pipes were known with any degree of surety, it was determined to start exploration in shallow water on the East side of the river, where the pipe was known to have only a few feet of cover. This plan served two purposes, it allowed for relatively quick test pit excavation to obtain projected pipe depth, and the shallow water allowed for better visibility, making identification and inspection of the pipe easier. Conversely, soil conditions in this area turned out to be relatively stiff marine clays, which made uncovering the pipes slow and difficult. The underwater slope characteristic of the soils in this area left the sidewalls of the excavation at a very steep slope. As such, while the initial penetration hole of the airlift took only a short time, it took a considerable amount of time for that airlift to widen the hole to a size where the diver could safely enter the hole. Because of these difficult soils the initial contact with the first pipe required the excavation of numerous "dry" holes, and took almost two days to accomplish. Once the pipes were finally found, and upon removal of the overburden from above the pipelines in the shallow trench, a diver equipped with a video camera entered the excavation and confirmed a pipeline had been located. In order to confirm the find, and determine if the diver was on the north, middle or south pipeline, personnel were positioned in the closest manhole that could provide land access to the pipes. The diver then "sounded" the pipe by tapping on the metal surface allowing the shore personnel to listen on each pipe for the tapping, then to report back to the barge. The diver then held a survey rod on the top of the pipe that extended above the water surface. The prism at the top of the survey rod was located by both land and water based surveys on shore utilizing a theodolite and electronic distance meter, a method that would provide the most accurate and reliable position data. The elevation of the top of the pipe relative to MLLW and its location relative to the Rhode Island State Plane coordinate system was then determined. A secondary redundant verification method was also used, wherein the location of the pipe was noted by the airlift positioning system and its depth was noted relative to the water surface elevation as read from the survey rod while it was held on the pipe.

Once an accurate position of each pipe was ascertained, it was possible to project each individually known pipe alignment ahead a few hundred feet to the next area of exploration, so as to reduce the search area for the next station. For the locations of the pipelines in deeper water it became necessary to locate a point on the cable suspending the airlift that was a known distance from the bottom of the airlift, then to place the air lift directly over the pipeline and shoot a hand held prism held on the mark with land based survey equipment. The secondary method used the distance from the known point to the water surface as measured by the water based surveyors from their survey boat, and the GPS location of the airlift.

Operations within the Federal Channel were more restricted than work outside of the channel, in that freight traffic navigating the channel had to be considered. At the end of each day's work, in consideration of maritime safety, the barge had to be moved from its working position, and relocated to a place near the edge of the channel out of the way of shipping. In addition, the excavation areas had to be surveyed at the end of each day, to ensure that any shoals crated by the airlifting did not protrude above the published navigation depth, where they could create a grounding hazard. Hydrographic surveys were also conducted during the excavation process to document the river bottom topography and to note the conditions before and after airlifting. This data was also collected to provide defense against potential shoaling claims from the dredging contractor.

Upon completion of the final pipeline location on November 2, 2004 the Contractor backfilled the open test pits that remained over the pipelines in the shallow areas.

Observations

The results of the pipeline survey were graphically presented on a plan prepared the Owner's Engineer. Field Engineers were on board the barge each day to position the barge and airlift as well as log the daily operations. Professional Hydrographic Surveyors also performed daily hydrographic surveys and provided supply and transportation between the barge and the shore. Copies of the daily survey data in the form of fathometer rolls and raw survey data were archived for future reference.

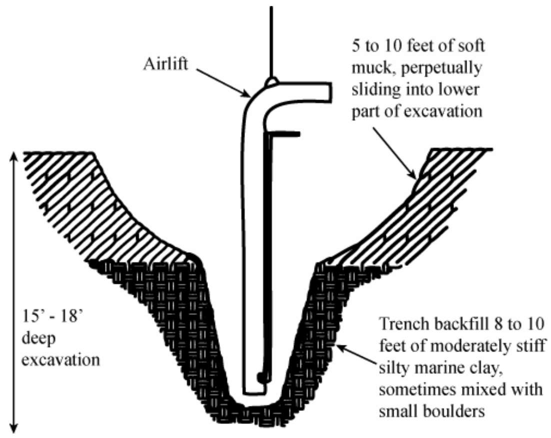

Fig. 4 Typical Air lifted exploration hole

In General most of the exploratory holes developed the funnel shape shown in the above figure. Soils on the Western limit of the channel were found to be considerably softer, and were unstable to deeper depths than the Eastern two thirds of the channel. The only way to overcome the instability was time-consuming, and involved the process of greatly enlarging the upper portion of the excavation. In other locations the stratification of the marine soils overlying the pipes caused a number of problems, including a persistent sloughing of the soft muck in the upper surface layers into the smaller lower excavation. At times a second soft layer was found near the pipeline depth, which tended to create a "bell hole" at the bottom of the excavation. This condition could only be stabilized by "punching" numerous holes through the intermediate stiff layer until it collapsed on itself, thereby opening the excavation to a width where it was safe for a diver to enter the excavation.

The pipeline inspection revealed a number of noteworthy conditions that differed significantly from prior documentation of record.

- In general the pipes were found to be at or close to the design grade of –50 feet MLLW, except in two locations. About 85 feet of pipe was found to be above the design grade on the Eastern shore, the shallowest pipe depth was at –44.7 feet below MLLW, or about 5.3 feet above the minimum permitted depth, in addition 25' of this section was above the elevation required to allow the minimum four feet of cover required for excavation of the new channel depth. Because of this encroachment, the Eastern 100' of the Federal channel was not dredged for the 150 foot length of the "no-spud" zone. On the Western limit of the Federal Channel about 95 feet of pipe was also found to be above grade, the shallowest pipe was found to be –49.7 feet below MLLW, or about 0.3 feet above the minimum permitted depth. This condition did not encroach on the minimum four-foot cover requirement, and thus did not affect the Federal dredging.

- In general, the 2004 electronic survey proved to be most accurate of the electronic location methods. In the western "no dredge" zone the actual pipeline depth was found to be approximately 1.5' to 3' higher than indicated by the 2004 Electronic Survey, which was close to the advertised vertical accuracy of ±2'.
- The 2004 Electronic horizontal position survey proved extremely valuable in locating the pipe and it proved to be the most accurate with respect to preceding methods, especially with respect to predicting the actual "bowed" alignment of the pipes. Further, because of the large disparity between the permitted pipe alignment and the actual alignment, if the 2004 Electronic alignment survey had not been available for initial guidance, pipeline location operations would have required considerably more time.
- While traces of gravel were found from time to time in the pipeline excavation, in no instance was the presence of measurable quantities of gravel backfill in the pipeline trench. (Gravel bedding and cover had been indicated on various plans and documents of record that were reviewed prior to the pipeline excavation.)
- The backfill material over the pipelines in the east side of the Federal Channel was found to be a moderately stiff marine silt and clay that held almost vertical side slopes. Conversely, the backfill material over the pipelines on the extreme West side of the Federal Channel was much softer (almost soupy), as such it would not hold reliable side slopes; this required that excavations be much larger. Concentrations of 12" boulders were found in a number of locations on the Western exploration area, which also made excavation more difficult. Not only did the boulders tend to lodge in and clog the air lift, but their presence made hand probing by the divers much more difficult.
- There was no evidence of typical marine coating on any of the pipes, such as heavy taping or concrete jacketing, nor was there any evidence of the cast iron jackets referred to in permit documents. Oak staves or strapping of unknown length, measuring roughly 2" by 2" were found along the long axis of a few of the pipelines in a few of the Eastern locations. The original intended purpose of this wood is not known.
- Two wooden piles were found protruding from the bottom in the vicinity of the first pipe excavation. It is assumed that these piles were either range markers or devices originally used to hold the pipes on line while they were being pulled across the river.
- The pipes were found to be out of compliance with respect to the 10-foot separation shown on permit drawings. The middle pipe was found to encroach on the

southern pipe, and in fact was laying directly on top of the southern pipe for a significant percentage of the route. It is possible that the middle pipe actually crosses over the South pipe somewhere on the eastern third of the channel, then re-crosses back over the South pipe about 500 feet east of the Federal channel limit.

- The pipelines were found to be south of their design locations by as much as 50'. Based on projections it could be as close as 20' to the southerly limit of the Federally designated "No Spud Zone".

The figures below show typical trench conditions that were anticipated from documentation of record, versus what was actually found from field exploration.

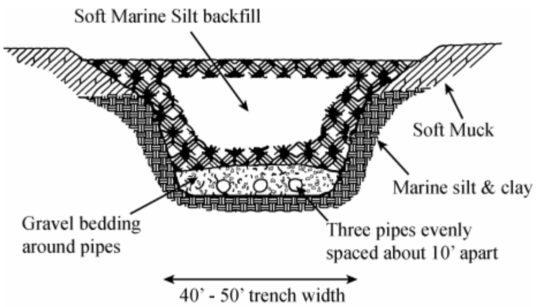

Fig. 5 Expected pipe placement and burial conditions based on documentation.

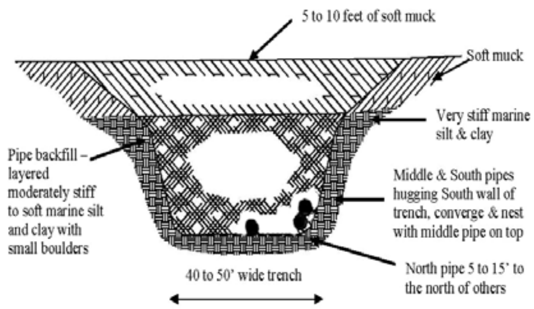

Fig. 6 Typical pipe trench conditions found during investigation.

Conclusions

Based on the results of the gas pipeline survey of October 2004, it was determined that the gas pipelines in the Western "no dredge" zone, were found to be much deeper than the earlier electronic surveys, yet still above the minimum design depth at several critical locations. The survey also indicated the gas pipelines had been placed well off of the planned alignment, which not only made them more difficult to locate, but nearly placed them in an unprotected dredge zone. The most non-compliant area was the Eastern most section of the pipe within the Federal channel. The shallowest pipe depth in this area was above the elevation required to allow the DOT's minimum four feet of cover required for excavation of the new channel depth. Because of this encroachment, the dredging of the Eastern 100' of the Federal channel was delayed until a safe method for its excavation could be worked out and approved. Ultimately the USACOE cleared the channel to the design depth with the provision that the interfering pipes ultimately be replaced or abandoned within the next few years.

In summary, this study emphasizes the importance of thorough and accurate survey, as well as careful documentation of underwater utilities placed in navigation channels. It also shows that one cannot always rely of older, uncertified plans of record, especially when inconsistencies are found. The impact of not doing so on this project was a delay in completion of the channel of at least four months, and a cost to the utility company of almost two million dollars.

Assessing PCCP Transmission Mains

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Abstract

The Washington Suburban Sanitation Commission (WSSC) has over 400 miles of prestressed concrete cylinder pipe (PCCP) in its transmission network for potable water, 150 miles of which is larger than 36 inches in diameter. WSSC has recently undertaken a condition assessment of several of their large diameter PCCP transmission mains, including the 96-inch Potomac main and the 60-inch North Adelphi Main.

The assessments relied on the application of electromagnetic inspections combined with visual and sounding inspections and acoustic monitoring for the North Adelphi Main. Following the inspections, pipes deemed to be in a state of incipient failure were excavated to confirm the findings and repaired as needed.

The focus of the inspections was to assess the condition of the prestressing wire for each pipe section, the structural component that provides the strength to PCCP. Electromagnetic inspections were used to test the condition of the wire. This type of inspection requires entering the pipe and traversing its full length with equipment that is capable of electromagnetically testing the prestressing wire wraps. The technology detects breaks in the prestressing wire and estimates the total number of breaks for each pipe section.

Visual and sounding inspections are a reliable method of detecting pipes in an advanced state of distress. The visual portion of these inspections looks for indications of distress resulting from a lack of prestressing. The sounding portion of these inspections rely on impacting the interior surface of the pipe wall with a tapping rod and listening for hollow sounds indicative of a lack of prestressing.

The inspection of the 96-inch Potomac main identified two pipe sections that required immediate repair. Excavations of these pipe sections revealed that the prestressing wire for these pipes was heavily corroded with numerous wire breaks. These

identification and replacement of these pipe sections most likely avoided a catastrophic failure.

For the North Adelphi main, following the inspection, a fiber optic sensor was installed in the main to permit continuous acoustic monitoring. The main was returned to service and is being continuously monitored to detect breaks as they occur in the prestressing wire. These wire breaks will be added to the wire break estimates from the electromagnetic inspection to update the wire break estimates for each pipe section. Using this data, the remaining useful life for each pipe section can be estimated.

WSSC's PCCP Assessment Procedures

Given the reliance of WSSC's water transmission system on large diameter PCCP mains, WSSC has long been a leading water utility in the assessment and management of PCCP. After years of research and development, which is still ongoing, for the two assessment programs covered in this paper, WSSC implemented the following assessment procedures:

1. Electromagnetic Inspection
2. Visual and Sounding Inspection
3. Acoustic Monitoring

All of these techniques provide information on the condition of the prestressing wire, the component that provides the pipe's strength and can be vulnerable to corrosion. The application of multiple technologies in one assessment project addresses the limitations of each technique when used on its own.

Assessment of 96-Inch Potomac Main

The 96-Inch portion of the Potomac transmission main consists of PCCP installed in 1967 to 1968 and has failed three times in the past, including at least one rupture with a loss of service. In 2006, a fourth failure occurred when a large leak was detected. Immediately thereafter, WSSC shut down the pipeline to repair the leak and to make it available for an internal inspection. The length of main inspected consisted of approximately 6300 feet of pipe.

The internal inspection consisted of a visual and sounding inspection and an electromagnetic inspection. The visual inspection involved traversing the pipe to visually inspect for evidence of a loss of prestressing a pipe section (e.g. longitudinal cracks consistent with wire break damage) and other problems that may be apparent. While this inspection was performed, a sounding inspection was also performed. During the sounding inspection the interior pipe wall was impacted with a tapping rod to listen for hollow sounds associated with a delamination. Delaminations are also an indication of a loss of prestressing in a pipe due to wire break damage.

Electromagnetic inspections involve traversing a pipeline with equipment that induces a magnetic field on one side of the pipe. The prestressing wire acts similar to an antenna and transmits the magnetic field to the opposite side of the pipe where it is carefully measured. The electromagnetic signature for each pipe section is evaluated in detail to identify the presence of electromagnetic anomalies consistent with wire break damage in a pipe section. These anomalies are further evaluated to estimate the extent of damage in the pipe including the estimated number of wire breaks.

The internal inspection identified two pipe sections that were deemed to be in a state of incipient failure. Given that the inspection was performed during a period approaching high demand (June 2006), WSSC executed and emergency repair contract to replace both pipe sections.

The first pipe section, numbered B-106, was found to have water leaking into the barrel of the pipe and an electromagnetic anomaly consistent with 80 wire breaks. This was the pipe section that was found to be leaking and was the reason that the main was shut down. Therefore, the finding of its condition was not a surprise. The pipe section was excavated under an emergency contract and replaced. Upon exposing the pipe section, an area of wire break damage was observed consistent with the location of the electromagnetic anomaly.

The second pipe section, B-175, exhibited an electromagnetic anomaly consistent with 100 wire breaks. It also had a 51 inch longitudinal crack on the internal pipe wall that was consistent with the types of cracks generated when a pipe section has lost prestressing. Given the both indications of damage on this pipe section, it was recommended that it be replaced prior to putting the main back in service. WSSC immediately searched for and purchased a repair kit, which was installed under the emergency contract. Upon exposing the pipe section, an area of wire break damage was observed consistent with the location of the electromagnetic anomaly.

Due to the emergency nature of these two pipe repairs a full forensic investigation could not be performed. The focus of these repairs was to repair the pipe and return it to service as quickly as possible. In addition to these two pipe sections, there were 44 other pipe sections that had electromagnetic anomalies consistent with wire break damage. Wire break estimations on these pipe sections ranged from 5 to 50 wire breaks. Since there was no indication of a loss of prestressing in any of these pipes based on the visual and sounding inspection, the main was returned to service.

Assessment of the 60-Inch North Adelphi Transmission Main

The North Adelphi Transmission Main consists of 47,300 feet of 60-inch PCCP main constructed in 1967 and 1968. The pipe was supplied by the Interpace Corporation and has failed multiple times in the past. The main was last inspected in 1995 and at that time one pipe section was identified to be in a state of incipient failure based on a visual and sounding inspection. It was repaired and the main was returned to service.

During the 2006 inspection, an inspection program similar to that used on the Potomac Main was implemented, but included additional tasks. The following tasks were completed:

1. Forensic investigation of a pipe section
2. Electromagnetic calibration on aboveground sections of pipe
3. Electromagnetic inspection
4. Visual and sounding inspection
5. Finite element modeling
6. Engineering recommendations
7. Limited forensic investigation on replaced pipe section
8. Acoustic Monitoring of the entire main

Forensic Investigation of Pipe Section: A section of the South Adelphi Main had previously been replaced as a result of a previous inspection performed by another consultant. A forensic investigation of this pipe section was implemented to determine the actual level of distress in the pipe. The first step of the forensic investigation was to chip a strip of mortar along the top of the pipe to expose the prestressing wires. A resistance meter was used to measure the resistance of each prestressing wire wrap to determine if the wrap was broken. Based on this testing, the pipe section had 28 wire breaks. To validate this finding the pipe was fully dissected. It was cut with a demolition saw along the top and all the coating was removed to expose the prestressing wire. The number of wire breaks was physically counted and a total count of 28 was observed, validating the results of the resistivity testing.

Electromagnetic Calibration on Aboveground Sections of Pipe: When feasible, it is advisable to perform a calibration of the electromagnetic inspection equipment on the pipeline to be inspected. Calibration involves cutting a known amount of prestressing wire wraps on a pipe section and performing an electromagnetic test to determine the electromagnetic response to a known level of damage in a pipe section. Numerous wire cut scenarios are created and electromagnetic signatures are obtained for each of the scenarios. This type of calibration provides the most accurate reliable electromagnetic inspection results.

Electromagnetic Inspection: In August and September 2006, Pure Technologies performed an electromagnetic inspection of the North Adelphi Transmission Main. The inspection identified 65 pipe sections that exhibited electromagnetic anomalies consisted with wire break damage. The estimate in number of wire breaks on individual pipe sections ranged from 5 wire breaks to 60 wire breaks. Two pipe sections, Pipe G296 and Pipe B217, were estimate to have 60 wire breaks. Pipe B28 and B153 were estimated to have 30 wire breaks. The rest of the pipe sections were estimated to have lesser amounts of damage.

Visual and Sounding Inspection: While the electromagnetic inspection was performed, Openaka performed a visual and sounding inspection. Most of the main appeared to be in good condition. However, some minor cracking was observed on seven pipe sections. Furthermore, one pipe section, Pipe G296, exhibited a 14" x 6.5" hollow area and multiple longitudinal cracks.

Finite Element Modeling: Since, based on the electromagnetic testing, a number of pipe sections were likely to have wire break damage. A structural analysis of the pipe was performed using finite element modeling. The model indicated that for one class of pipe with 13.5 feet of cover, 60 contiguous wire breaks would likely cause the pipe to fail. The two pipe sections estimated to have 60 wire breaks consisted of pipe from this class.

Engineering Recommendations: Pipe G296 exhibited an electromagnetic anomaly consistent with 60 contiguous wire breaks and a hollow area with cracking. As a result, this pipe section was immediately recommended for repair. Pipe B217 was estimated to have 60 wire breaks, but did not exhibit any signs of distress during the visual and sounding inspection. However, given that the estimated wire break damage was approaching the point of failure based on the structural analysis this pipe section was also recommended for repair. Both of these pipe sections were replaced. The remainder of the transmission main was determined to be at acceptable levels of risk.

Limited Forensic Investigation on Replaced Pipe Sections: While the contractor was removing Pipes G296 and B217, a limited forensic investigation was performed. The goal of the forensic investigation was to ascertain an actual number of wire breaks, but not to impede the contractor.

For Pipe G296, it was determined that there was 57 wire breaks. The wire appeared black in this region and the mortar coating was softened and thin, consistent with aggressive groundwater attacking the pipe. The expected length of damage and the number of wire breaks estimated by the electromagnetic inspection were consistent with the electromagnetic testing results.

For Pipe B217, upon excavation the same conditions appeared to be present for this pipe section. An area of black wire and thin mortar was observed a 5 foot long section of the pipe section. Within this area of corrosion there were two wire break zones, with a total of 21 to 26 wire breaks. The first wire break zone consisted of 16 contiguous wire breaks and the second zone consisted of between 5 and 10 wire breaks. The two wire break zones were separated by a longitudinal distance of 1.0'. The expected zone of wire break damage based on the electromagnetic inspection was 2.2' long. The expected number of wire breaks, calculated based on this length, was estimated to be 60 wire breaks. The length of damage correlated well with the actual conditions (2.2' vs. 2.0'). The number of wire breaks was in the correct order of magnitude, but was overestimated due to multiple wire break zones in close proximity.

A table comparing the forensic results of Pipe B217 and G296 is provided below.

Table 1
Electromagnetic Inspection Results vs. Observed Conditions

	Estimated Based on Electromagnetic Inspection	Actual as Observed in Field
Pipe G296 Number of Wire Breaks	60	57
Pipe G296 Estimated Length of Wire Break Damage Zone	2.6 feet	2.5 feet
Pipe B217 Number of Wire Breaks	60	21 to 26
Pipe B217 Estimated Length of Wire Break Damage Zone	2.2 feet	2.0 feet

Acoustic Monitoring of Entire Main: WSSC opted to install an acoustic fiber optic sensor in the full length of the main. The fiber optic sensor will be used to monitor the acoustic activity in the pipeline to track wire breaks into the future. Wire breaks detected by the acoustic monitoring system can be added to the estimates provided through the electromagnetic inspection to keep track of the condition of each pipe section. If pipe sections, based on the total number of wire breaks, approach a point of undesirable risk, repairs can be proactively implemented.

Conclusions

The Potomac and North Adelphi Transmission Mains were inspected utilizing electromagnetic inspection and visual and sounding inspection techniques. A total of four pipe sections were replaced based on the inspection results. Each pipe section was confirmed to have significant levels of corrosion damage consistent with the inspection results. The remainder of both transmission mains was at acceptable levels of risk and returned to service.

It is likely that both inspection projects avoided the catastrophic failure of these large diameter high-pressure transmission mains. As a result, it is clear that proactive assessment and management of transmission mains can extend the service life of transmission mains and avoid outages as a result of unexpected failures.

By combining assessment techniques, WSSC was able to obtain a reliable assessment of both mains. By the time ground was broken to implement repairs, all parties were reasonably confident of the condition of each pipe section. When implementing repairs on pipes of this diameter it is important to establish this level of confidence.

CONDITION ASSESSMENT OF AN ASBESTOS CEMENT PIPELINE

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ABSTRACT

The Greybull Water Transmission Pipeline (GWTP) was constructed as a steel line in the 1930s, but was replaced in late 1960s and early 1970s with 12 in. and 14 in. diameter asbestos cement (AC) pipeline because of steel corrosion and numerous leaks. Greybull, along with many other towns and municipalities, are faced with projected increase in future demand and the need to determine if the current infrastructure can safely and adequately meet the future demands.

The condition assessment study was performed to determine if the line can continue to provide service under the proposed higher flow conditions. The scope of the work included review of available data, structural evaluation of the pipeline, determination of corrosivity of soil and ground water, and internal water to AC pipe, excavation and external inspection of the pipe, testing of AC pipe material, including three-edge bearing test, flexural test, and petrographic examination of the pipe wall.

The results indicate that the AC pipe has not deteriorated significantly over the past 35 years, except minor to moderate acid attack that affected up to about 8% of pipe wall thickness in sporadic locations. There is no evidence of sulfate attack despite high soil sulfate content over most of the line, and of any other systematic deterioration mechanism. Structural evaluation for combined loads indicated that the pipe has adequate factor of safety and can continue to provide service under the proposed hydraulic modifications.

1. INTRODUCTION

Use of AC pipe for water mains started in the 1930s. AC was extensively used between the 1950s and 1980s, until the health concerns with asbestos significantly reduced and eliminated the use of AC pipes in the US. History of AC pipe performance in other water systems has shown that AC pipe typically fails as a result of stresses and strains in the pipe wall exceeding the strength of the pipe, which is reduced as the pipe deteriorates by internal or external chemical attack. The failure modes of AC pipe include longitudinal cracking, circumferential cracking, joint failures, deterioration and loss of wall section resulting in holes in the pipe wall. Deterioration is typically caused by external or internal corrosion from acidic soils or water, and from high soluble sulfates in soil and water. Smaller diameter pipes are more prone to failures resulting from longitudinal stresses, due to differential settlement, than from pressure. Studies in England and Canada indicate AC pipe failure rates of about 6 to 10 breaks per 100 kilometers of pipeline per year of service (or 9 to 16 breaks/100 miles of pipeline per year) and increasing with age of the pipe.

The Greybull Water Transmission Pipeline (GWTP) provides water to the town of Greybull and individual users along approximately 16.5 miles between the Shell Creek wells and the 1 MG reservoir near town. Originally, the pipeline was constructed as a steel line in 1930s, but was replaced in late 1960s and early 1970s with asbestos cement (AC) pipe, because of steel corrosion and numerous leaks. Parts of the pipeline were replaced with 12 in. diameter Class 150 AC pipe in 1960s. The remaining parts of the pipeline were replaced in 1973 with 14 in. diameter Class T40, T45, and Class T60 AC pipes. Performance of AC pipe in GWTP to date has been good with only one reported pipe failure in 1999 during a transient event and another instance where a pipe was damaged during excavation for other utilities and had to be repaired. With only two failures, including the failure resulting from the damage inflicted during an excavation for another utility, the failure rate of GWTP has been about 0.34 breaks/100 miles per year, significantly lower than the rates reported elsewhere.

1.1 Purpose and Scope

The purpose of the study was to perform condition assessment of the GWTP asbestos cement pipeline.

The scope of work included review of available data, structural evaluation of the pipeline subjected to the combined effects of external loads, internal pressure, and pipe and fluid weights, determination of corrosivity of soil, ground water, and internal water to AC pipe, excavation and external examination of the pipe, mechanical testing of pipe, and petrographic examination of samples of AC pipe material.

2. AC PIPE DESIGN

AC pipe is made from a mixture of Portland cement (or cementitious materials) and asbestos fibers with or without silica. The material is formed under pressure and cured to meet the physical and chemical requirements of the standards. AC pipe materials, manufacture and design are specified in AWWA C400 to C403 and ASTM C296, C500 and C688 standards.

AC pipe is designed for combined internal pressure and external loads based on the minimum hydrostatic pressure strength and three-edge bearing load using a factor of safety to calculate the allowable pressure and earth load. AWWA C403-78 Standard for Design of Transmission pipelines, such as GWTP, recommends the use of a minimum factor of safety of 2.0 for working plus transient pressure, a factor of safety of 1.5 for external soil and live loads, and a factor of safety in between the two as determined by an empirical interaction formula for combined loads.

3. EXTERNAL INSPECTION

External inspection of the pipe was performed at ten sites to examine the condition of the pipe, collect samples of soil and ground water, perform non-destructive testing on the pipe, and to remove samples of pipe for testing and analysis.

Excavation sites were selected based on the review of local geology, soils, topography, and preliminary results of hydraulic analysis and structural analysis of the pipe, such that the selected pipes were located in some of the most corrosive soils to AC pipe along the pipeline, and in other baseline conditions. Soils corrosive to AC pipe typically have high concentration of sulfates, low pH, and high or variable groundwater table in the pipe zone. We also identified areas with clayey soil that may result in differential settlements, and areas of high internal pressures. We also selected baseline sites in dry sandy soils.

Sample size of ten sites is adequate for diagnostic purposes to determine general condition of the pipeline and if there are systematic problems. Obviously, sample size is too small to determine probability of failure or to arrive at conclusions related to the reliability of individual pipe pieces.

External inspection consisted of the visual observations of the pipe exterior and soil in the trench, collection of soil and ground water samples, Ultrasonic Pulse Velocity (UPV), and Schmidt hammer non-destructive tests. The nondestructive tests were performed to measure the relative differences in stiffness of the pipe wall. The Schmidt hammer measures surface hardness to locate areas of surface deterioration and the UPV measures the pipe wall deterioration by measuring the UT wave velocity, a function of the dynamic elastic modulus and density of AC. The results of external inspections of the pipeline indicate the following:

- No cracks, delamination, hollow sounding, or other anomalies were observed on the pipe at any of the excavation sites. The pipes have a waffle like finish as a result of rolling operation used in fabrication. This finish was still visible in all excavated pipes to varying degree (Photo 2).

- Ultrasonic pulse velocity indicated relatively uniform stiffness with no significant anomalies in any of the inspected pipes.

- Schmidt hammer results indicated some variation in pipe wall surface stiffness. The lowest readings correlate with observed surface deterioration in petrographic analysis (see discussion below).

- Soil in the excavations ranges from clay/silt to sand with some clay/silt. Groundwater was observed in half of the excavations.

Photo 1 - Surface of exposed pipe and joint.

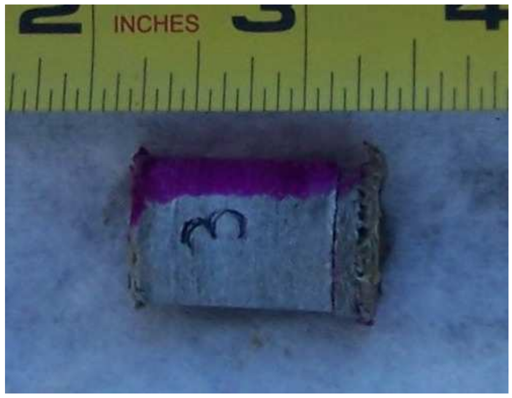


Photo 2 – Tap Sample

4. MATERIAL TESTING

The purpose of material testing was to check the quality and condition of asbestos cement pipe and corrosiveness of the environment. We performed the following tests:

- Mechanical testing of AC pipe.
- Soil sieve analysis and classification.
- Petrographic examination of AC pipe.
- Chemical analysis of soil and ground water in excavations.
- Chemical analysis of water in the pipeline.

4.1 Mechanical Testing

A piece of 14 in. diameter pipe, Class T40 (most prevalent and the lowest strength pipe class) was removed from the line for testing. We performed the following tests on pipe samples in plastic bags:

- Three edge bearing test (i.e. the crushing test specified in ASTM C500) to determine whether the pipe has sufficient strength to withstand the crushing load required by ASTM C688 (Photo 3) standard, and
- Four-point bending test of 2 in. wide longitudinal strips of pipe to measure longitudinal flexural strength of the pipe wall.

The results indicate the following:

- The three three-edge bearing strength was between 6,621 lb/ft and 7,032 lb/ft, significantly higher than the minimum crushing strength of 3,000 lb/ft specified in ASTM C668 for 14 in. diameter Class T40 pipe.

- The average of the longitudinal flexural strength measured in five flexural strength tests were 4,561 psi.

Photo 3 - Three-edge bearing strength test of 14 in. diameter AC pipe.

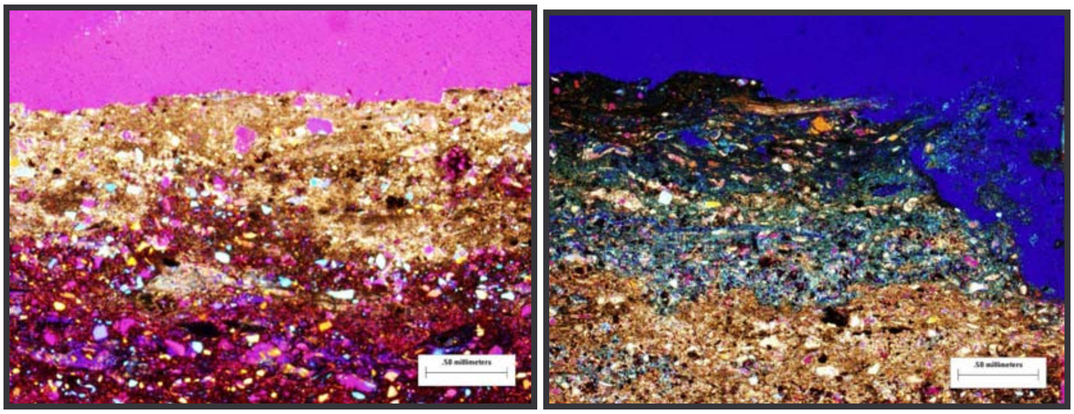
4.2 Petrographic Analysis

Samples of AC pipe for petrographic analysis were obtained by tapping the pipe at six locations. Petrographic analysis of samples of AC pipe wall were performed in accordance with ASTM C856 on polished and thin sections of AC pipe wall.

The results of petrographic analysis indicate that the overall composition and microstructure of asbestos cement is very good with very low water-to-cementitious material ratio (<0.35), no entrained air, and uniform asbestos fiber distribution (Photo 4). We observed carbonation (ranging in depth from 0 to about 3 mm) and bicarbonation of the paste (ranging from 0 to about 2 mm) (Scali 2003). No evidence of sulfate attack was observed in any samples. Minor to moderate acid attack was observed on the exterior surface of two samples. Depth of physical alteration of the paste (effective reduction in cross-section) that may affect strength of the pipe wall was less than 1 mm (less than 2.6%) in all but one sample, where it was 1.2 to 1.8 mm (up to 8.6% of cross section). This sample also exhibited the lowest Schmidt hammer measurement of all test sites, indicating the lowest surface stiffness.

4.3 Soil and Groundwater Testing

For asbestos pipe, the primary mechanisms of potential chemical degradation from aggressive chemicals in the surrounding soil are sulfate deterioration, concrete carbonation, and acid attack. The chemical analysis of the soil and groundwater from locations adjacent to the pipe indicate that there are two broad categories of soil chemistry conditions along the pipeline, with either significant or low to trace amounts of leachable calcium, sodium, and sulfate ions in the soil and groundwater:



(a) (b)

Photo 4 – Magnified (25X) view of (a) outer surface of Sample 4 (left) showing very minor alteration and discoloration of surface paste structure with no distress or loss of surface, and (b) outer surface of Sample 11 (right) showing minor to moderate evidence of paste alteration and increased porosity of the surface paste (blue color) as a result of acid attack.

Sulfate Deterioration. The high levels of soluble calcium, combined with the high levels of soluble sodium in an area indicate that the area contains significant amounts of soluble sodium sulfate. Soluble sodium sulfate can induce sulfate deterioration in concrete. The rate of attack is controlled by the permeability of the concrete, absorption rate of water, and the diffusivity of the sulfate ion into the concrete, as well as the availability of calcium hydroxide and calcium aluminate in the concrete to react with the sulfate ion to form expansive reaction products.

AC pipe is typically manufactured to ASTM C500 specifications of either Type I or Type II pipe depending on the level of sulfate resistance. Type II pipe is resistant to all levels of soluble sulfates, has less than 1% uncombined calcium hydroxide and is considered chemically resistant pipe, when tested according to the method given in ASTM C500.

Our petrographic examination of the pipe did not find sulfate induced deterioration. In addition, we observed very little free calcium hydroxide. Therefore, the pipe is likely Type II, low calcium hydroxide content, sulfate resistant pipe, as confirmed by the manufacturers stamp found on a pipe in one excavation.

Concrete Carbonation. Concrete carbonation reduces concrete pH and may increase the potential for steel corrosion, but if there is no reinforcing steel, such as in AC pipe, carbonation may be beneficial by providing denser less permeable concrete. Although we observed carbonation in the exterior surface of the concrete pipe samples, from o mm to 3 mm deep; we did not observe any deterioration caused by carbonation.

Acid Attack. In general, concrete is not resistant to acid attack and it can also degrade in the presence of bicarbonates at neutral pH's. The very low degree of saturation in Sample 11 soil (24%) indicates a potential for the soil becoming acid, especially during ingress of acidic surface water (rain water may have a pH of about 5). In addition, Soil Sample 11 contains a minimal amount of soluble calcium, sodium, and sulfate (Condition 2 soil), which indicates a lesser ability for the soil to buffer acidic surface water. Thus, although the pH of Sample 11 soil is 8, the soil can allow deterioration of the concrete due to acid attack from passing lower pH water. We found deterioration of calcite aggregate particles and porous concrete in the outer 1.2 and 1.8 mm of the AC in the near-surface region of Sample 11, as indicated by hollow particles and re-precipitated crystals of calcite/aragonite along the particle edges. Both of these conditions are consistent with minor to moderate acid attack of the concrete in Sample 11.

4.4 Aggressiveness of Water in the Pipe

The results of testing of water in the pipe indicate that the water is not aggressive to AC Pipe as measured by Aggessivity Index (AI) of 13 than 12.0 considered as non-aggressive in ASTM C500.

4.5 Asbestos Fibers in Water

The test for presence of asbestos fibers in the water indicated that there are no asbestos fibers in the water.

5. HYDRAULIC ANALYSIS

The pipeline underwent several changes in operation since installation in 1930's. Near the wells at upstream end of the pipe, there are local water storage tanks that provide chlorine contact time as well as demand buffering for peak demands on the transmission line. The current hydraulic water line is at elevation 4,383 ft, and the low elevation in the pipeline is at about 3,900 ft. There are pressure reducing and sustaining valves (PRV), Roll-Seal style valves, along the pipeline. The pipeline is typically operated by occasionally adjusting the valves along the pipeline.

In the present operation of the pipeline, the flow is never stopped. Stopping the flow would result in zero flow hydraulic grade line at the elevation of 4,383 ft, which could increase the maximum HGL by up to about 70 ft of hydraulic head and could result in significantly higher pressures (up to about 225 psi) than pipe design pressures. Sudden closure of valves could result in even higher pressures of about 300 psi that could damage and even rupture the pipeline. It appears that the only pipe failure to date occurred during a transient pressure event caused by rapid valve closure.

Future operation of the pipeline envisages increase in flow and control of flow to minimize spillage from the tank. This would require modification of the valve system along the pipeline, to maintain the maximum transient pressure below 145 psi as discussed in the structural evaluation section below.

6. STRUCTURAL EVALUATION

Structural safety of the AC pipe was determined along the pipeline based on pipe class, maximum working and working plus transient pressure, and earth load. AC pipe is designed for combined internal pressure and external load following the empirical earth load-pressure limit expressed by a parabola where horizontal and vertical axis intercepts are the burst pressure, and the ultimate three-edge bearing load, respectively. For the three-edge bearing load, we used 6,000 lb/ft, the measured three-edge bearing strength for Class T40 pipe (the design load is given as minimum 3,000 lb/ft, and the actually measured range of crushing strength is between 6,621 lb/ft and 7,032 lb/ft).

Earth load was calculated based on cover height of 6 ft, based on the measured soil cover in excavations ranging between 5 and 6 ft. Obviously, in areas where the soil cover is lower, the factor of safety is higher, and where soil cover is higher, the factor of safety is lower.

The ultimate and allowable envelopes for working and transient condition, were calculated using factors of safety recommended in AWWA C403 for transmission pipelines, i.e., 2.5 for burst pressure, and 1.5 for earth load. The ultimate and allowable transient combined load envelopes are plotted in Figure 1, together with the data points for the maximum working and transient pressures for Class T40 pipe and soil plus live load for 6 ft of cover, and the factor of safety lines depicting an intersection with the combined load ultimate strength used to calculate the factor of safety.

The results of our structural analysis, based on the minimum design hydrostatic pressure strength and measured three-edge bearing strength indicate the following:

- For the current maximum working pressure in Class T40 pipes earth and live loads of 6 ft of cover, the factor of safety is well within the design envelope. If the flow in the pipeline was stopped by closing the valve at the tank slowly, this would result in reduction in safety and the design would be significantly outside of the design envelope. Naturally, if there was a rapid closure of the valves, this would result in higher pressures, and even lower factors of safety.

- For the future maximum pressure after modification of PRVs, the design would lie only slightly outside of the transient envelope specified by AWWA C403.

- Assuming future maximum pressure condition and loss of thickness by 10%, determined from observed loss of thickness in pipe petrographically examined, the pipe would have a factor of safety meeting the design envelope over most of the line (Figure 1), and no point would be more than 15% below the recommended factor of safety by AWWA C403 (Figure 2).

- To protect the AC line from excess surges, future changes in flow need to be designed to keep transient pressures low enough without exceeding the design envelope.

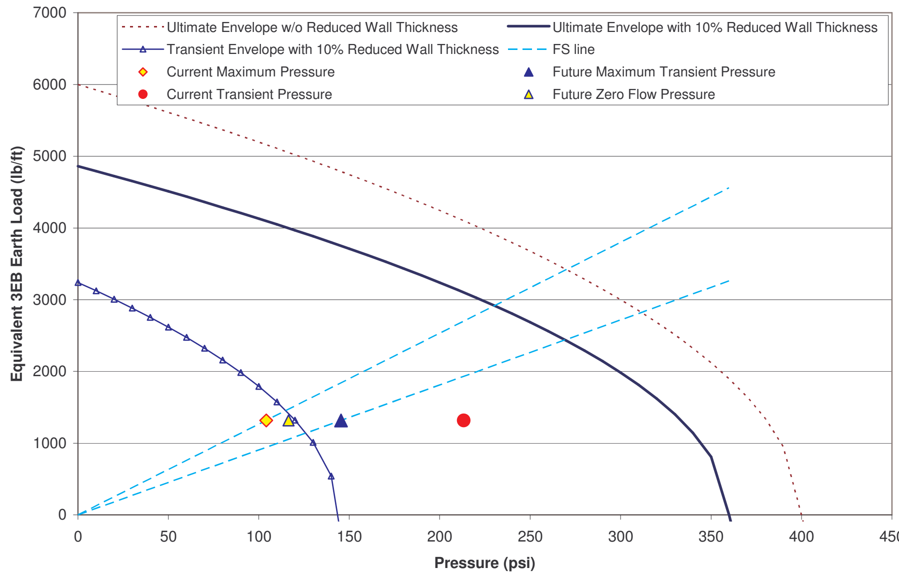


Figure 1 – Earth Load-Maximum Pressure Envelopes for AC Pipe Class T40 with Reduced Wall Thickness by 10%, and with Maximum Pressure Point and Factor of Safety Line

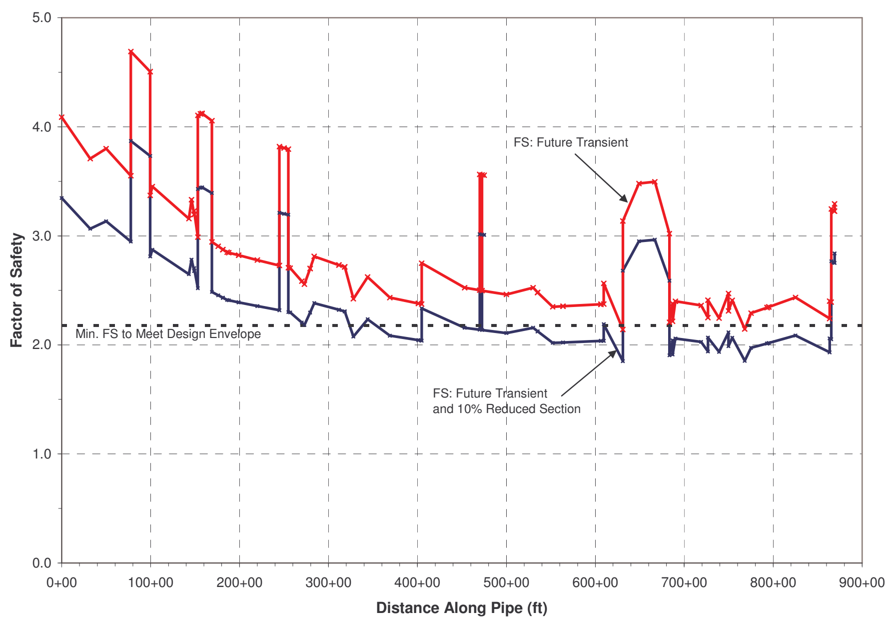


Figure 2 – Future Pipeline Operation Factor of Safety for an Assumed Soil Cover of 6 ft

7. CONCLUSIONS

Based on the results of exterior inspection and material testing, petrographic and chemical analysis, hydraulic surge analysis, and pipe structural evaluation we concluded the following:

- The failure rate of this pipeline has been much lower than the rates published for other AC lines in literature. We have no reason to believe that the failure rate of this pipeline would not continue to be lower than that of other similar AC pipelines.

- The pipes have not deteriorated significantly over the years, as evidenced by only minor to moderate acid attack in sporadic locations. We did not find evidence of sulfate attack on any of the samples of pipe examined, and no evidence of systematic deterioration. However, owing to the limited sample size, it is possible that there may exist pipes or areas with worse deterioration than that observed.

- GWTP line is safe to operate under existing working pressures, and may be operated under the proposed future increased flow condition after modifications to the PRV's to maintain the maximum pressure in the line below 145 psi, but with reduced safety. Un-deteriorated pipes will have a safety approximately equal to that recommended by AWWA C403. Deteriorated pipes (assuming 10% loss of section) will have safety up to 15% below that recommended by AWWA C403 over short areas of the pipeline.

Acknowledgement

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References

Scali, M., Scheiner, P., Aquino, W., Ojdrovic, R. P., and Zarghamee, M. S. (2003). "Analysis and Performance of PCCP Mortar Coating," *Proceedings of ASCE Specialty Conference Pipelines 2003, ASCE Specialty Conference,* Baltimore, Maryland

Analyses of Ductile Iron Corrosion Data from Operating Mains and Its Significance

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Abstract

This paper discusses actual experience with independent (by owners or their consultants) corrosion evaluations of ductile iron (DI) pipelines. Included are evaluations of the extent of corrosion on DI in various soil environments, the effectiveness of polyethylene encasement for corrosion control, the effectiveness of the standard factory applied asphaltic coating for corrosion control, and the effectiveness of the 10-Point Method in predicting corrosion activity on DI.

Introduction

Ductile iron (DI) piping has been used in the United States since the 1960's for water and waste water pipelines. By 1979, DI pipe largely replaced cast iron as the predominant pipe material in the water and waste water industries. Therefore, the majority of DI pipelines have only been in operation for 27 years or less. Failures of DI piping have become more prevalent as the pipelines have gotten older.

The failure of DI pipe due to external corrosion is not a new or recent phenomenon. Failures of DI pipe have been reported since the early 1980's[1-3], through the 1990's[4,5] and during the 2000's[6-11]. In fact, as the water industry gains knowledge regarding corrosion, the number of reported corrosion failures on DI pipelines is increasing. Unfortunately, even today, many failures on DI pipe are repaired by water department crews without being evaluated for corrosion and without documentation as to the cause of the failure. Therefore, the true extent of the problems associated with DI pipe corrosion is not known. However, it is evident from the information that has been published[6-11] and from experience, that corrosion failures on DI pipe are widespread and increasing.

There are a limited number of specific examples of independent DI corrosion evaluations in the literature. Therefore, a database of DI corrosion evaluations has been developed to document the experience of pipeline operators. The data have been compiled from a number of sources and include data from published technical articles as well as from independent corrosion engineers. The accuracy of all of the data that has been provided for this compilation could not be independently verified because the data were provided by others and were obtained from various evaluations over several years. However, every attempt has been made to accurately compile and present the data provided.

Data Collection Methods

The evaluations of DI pipe corrosion that are included in this analysis are based on reported actual experiences of operating pipelines. The evaluations include examples of corrosion penetrations of pipe walls as evidenced by leakage of water or waste water. In some examples, the perforations eluded discovery until the pipe was excavated and examined, based on surface measurements of corrosion potential during pipeline condition evaluations.

The following methodologies were used to identify the instances of external DI pipe corrosion included in the analysis:

- Cell-to-cell corrosion potential measurements correlated with soil resistivity data to locate active corrosion.
- Pipe excavated and exposed during repairs occasioned by a pipe failure.
- Observation of pipe, excavated and visible in areas targeted as sites of likely corrosion.

The results of evaluations in random test pits were not included in this database. Significant levels of external corrosion will typically impact 10% or less of an operating pipeline's surface. Therefore, the odds are only 1 in 10 that a random test pit would reveal an area of significant external corrosion activity. One example included in the database is the result of a pipe sample installed at a test site. The one example of a test site evaluation that was included in the database was obtained from a published technical paper[8] that reported that the depth of corrosion pitting on DI under undamaged polyethylene encasement was greater than the depth of corrosion directly at a deliberate tear in the polyethylene encasement. The other examples that have been included in this database include operating water and sewer mains.

Description of Database

The database consists of 60 examples of DI pipe evaluations. Figure 1 (at the back of this paper) shows a summary of the database. Figure 2 shows one of the pipe segments that is included in the database. The pipe sizes in the database range from 4 inches to 24 inches in diameter. Table 1 shows the number of examples for each of the different pipe sizes included in the database. The DI piping ranges from 5 to 35 years in age. Table 2 shows the number of examples in each age group.

Figure 2
27-year old DI pipe with no protection (after cleaning).

TABLE 1
Pipe Diameter Summary

Pipe Diameter (Inches)	Number of Examples
4	1
6	8
8	14
10	1
12	15
14	1
16	15
18	3
24	1
No Data	1

TABLE 2
Pipe Age Summary

Pipe Age (Years)	Number of Examples
Less than 10	4
10 to 19 years	18
20 to 29 years	26
30 years or more	12

The 60 DI evaluations include 31 examples of failed (penetrated) pipe. Thirteen of the 60 DI evaluations were of polyethylene encased piping. Seven of the polyethylene encased pipes included in the database were at failure locations.

Data Analyses

The average corrosion rate was calculated for each pipe segment by dividing the depth of corrosion by the age of the pipe. The average corrosion rate ranges from 3 to 68 mils per year. Thirty-five of the DI specimens had corrosion rates greater than 10 mils per year and 21 had corrosion rates less than 10 mils per year. Corrosion rate data were not available for 4 of the 60 DI examples. The number of examples within each corrosion rate category have been tabulated and are shown in Table 3.

TABLE 3
Corrosion Rate Summary

Corrosion Rate (mils/year)	Number of Examples
Less than 5	5
5 to 9.9	16
10 to 14.9	15
15 to 19.9	11
20 to 24.9	7
25 to 29.9	0
30 or Greater	2
No data	4

Soil corrosivity data were available from samples for 51 of the 60 DI corrosion examples. Most of the soil samples were reportedly obtained from the excavation at or near pipe depth. However, some of the soil samples were reportedly obtained near the excavation or from a depth other than pipe depth. Using the soil corrosivity data, the 10-Point Soil Test Evaluation Method[12] was applied to calculate the respective corrosivity of the soil for each example. According to the 10-Point Method, values below 10 points are "non-corrosive" and values that are 10 or greater are "corrosive". Some of the soil corrosivity evaluations did not include Redox potential. In those instances, the Redox potential was assumed to be negative which would represent "worst case" conditions.

The 10-Point Method calculation values range from 0 to 24 points. 26 of the values were less than 10 points ("non-corrosive") and 26 were 10 or greater ("corrosive"). Eight of the pipe evaluations included insufficient soil corrosivity data to calculate a 10-Point value.

Results of Data Evaluation

Actual Pipe Condition Correlated with the 10-Point Soil Test Evaluation Method

The 10-Point Soil Test Evaluation Method purports to provide an evaluation of soil corrosivity so that appropriate corrosion control measures can be implemented on a DI pipeline. By applying the 10-Point Method, soil is determined to be "non-corrosive" or "corrosive" and the recommendations include no corrosion protection for pipe in "non-corrosive" soil and polyethylene encasement in "corrosive" soil. The 10-Point Method makes no assertions regarding how long a pipeline may be free of corrosion failures in either category of soil corrosivity. However, the 10-Point Method does indicate that soil and other conditions will affect the rate of corrosion of DI pipe. The 10-Point Method does discuss "uniquely severe environments" that may require other corrosion control measures in place of, or in addition to, polyethylene encasement. None of the DI evaluations included in this database were in a "uniquely severe environment" as defined by the 10-Point Method. Rather, the pipelines included in this database were in soil categorized as "non-corrosive" or "corrosive" by the 10-Point Method in the instances where data were available for the calculation.

The data from the 60 DI pipe evaluations were evaluated to determine if the 10-Point Method provides a meaningful assessment of soil corrosivity. If it does, it would be expected that the rate of corrosion would increase as the value of the 10-Point Method goes up. Therefore, corrosion rate data were correlated to the 10-Point values. The data correlation is shown in Chart 1. The corrosion rate data do not correlate well with the 10-Point soil values as evidenced by the calculated correlation coefficient of 0.2898. A correlation coefficient of 1 would indicate a perfect correlation. The corrosion rate data for only the 24 failed pipe specimens with corrosivity data available were then correlated to the 10-Point values for those examples. The plotted data are shown in Chart 2 and indicate that the 10-Point values do not correlate to the rate of corrosion for the failed pipe specimens. Those data have a correlation coefficient of 0.4289. The 10-Point values were then correlated to the age of the failed pipe specimens. The age of the failed pipe included in this correlation ranged from 15 to 31 years and the 10-Point values ranged from 1 to 23.5. The plotted data are shown in Chart 3 and indicate no meaningful correlation (correlation coefficient of 0.28765) between the age of the 10-Point values and the age of the failed pipe specimens. The data clearly indicate that the 10-Point Method is not necessarily useful in the prediction of corrosion activity on DI piping. The findings of this evaluation are similar to the results of a study conducted by Wakelin and Gummow[13] on water mains in Ontario, Canada.

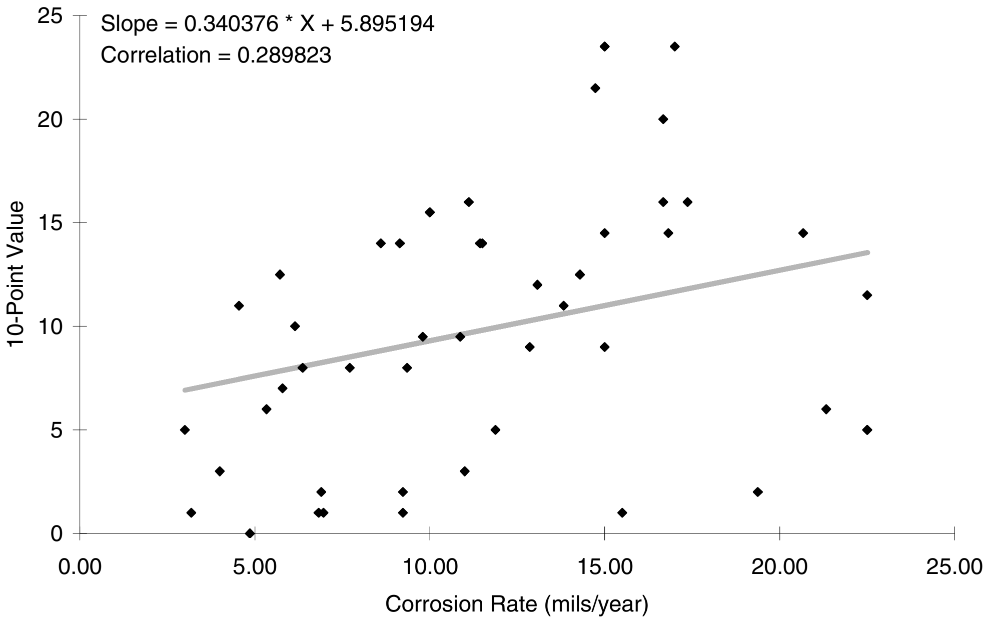

Chart 1: Corrosion Rate Versus 10-Point Value

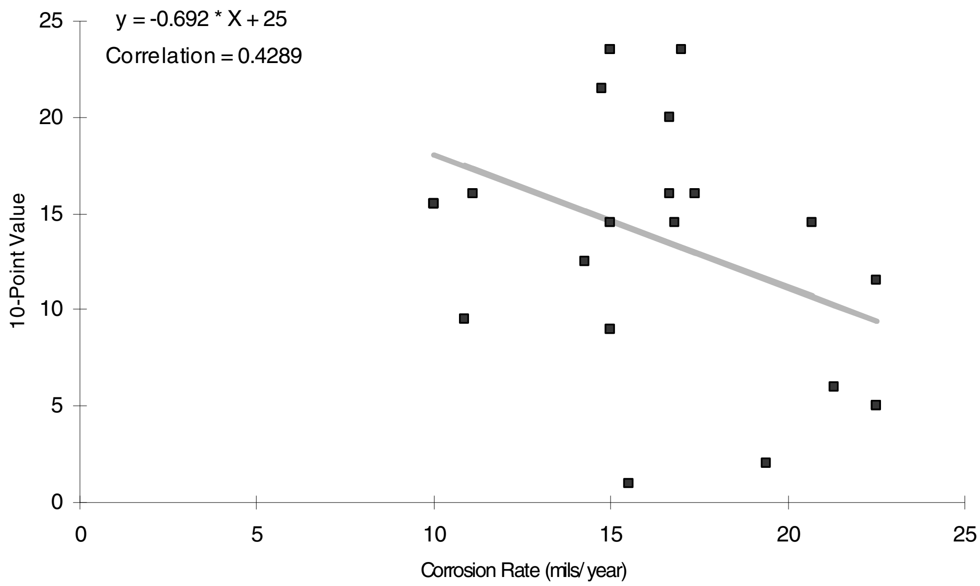

Chart 2: Corrosion Rate Versus 10-Point Value (Failed Pipe Only)

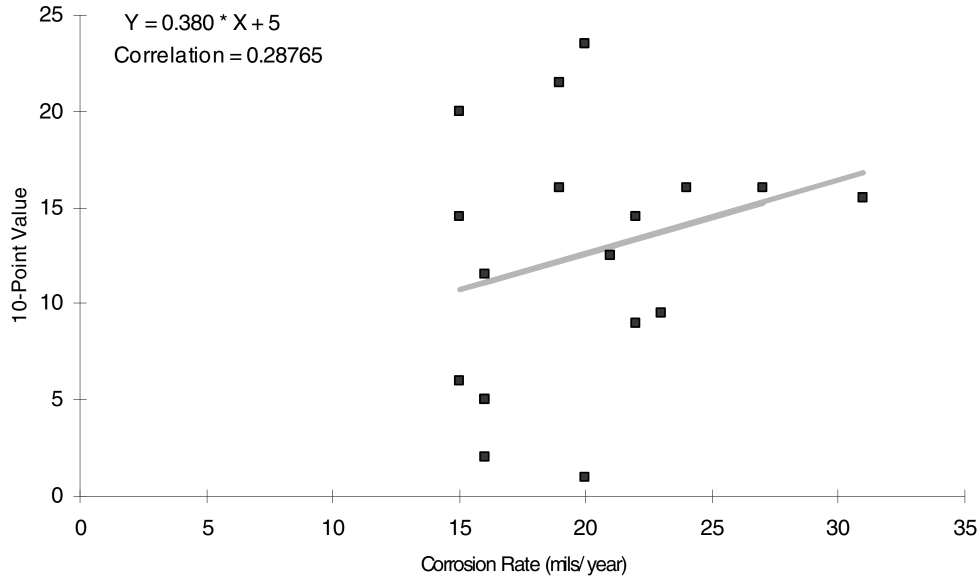

Chart 3: Pipe Age at Failure Versus 10-Point Value (Failed Pipe Only)

Effectiveness of Polyethylene Encasement for Corrosion Prevention

Polyethylene encasement (PE) has been promoted by the DI pipe industry for many years as the only corrosion control measure that is required for ductile iron pipelines that are exposed to "corrosive" soil as determined by the results of an analysis of soil samples by the 10-Point Soil Test Evaluation Method. This database includes the results of evaluations that were conducted on 13 polyethylene encased DI pipe segments. The 13 polyethylene encased DI pipe specimens range in size from 6 to 24 inches in diameter and include 7 failed pipes. The age of the polyethylene encased piping that was evaluated ranged from 5 to 20 years. In some cases corrosion occurred under intact PE at the same rate as corrosion that occurred at tears in the PE, in some cases the specific location of the corrosion and condition of the PE was not reported, and in one example from a DIPRA test site[8], it was reported that the depth of corrosion pitting was 150% greater under intact PE than it was where the PE was deliberately damaged. The examples of PE in this database indicate that PE was not helpful in preventing corrosion on DI pipe in many soil types.

Impact of Internal Pipe Linings

In 13 of the 31 examples of failed (penetrated) DI pipe, the internal lining (cement mortar or polyethylene) is known to have delayed leakage for some period of time. In those 13 instances, the internal lining bridged across the penetration until the penetration became too large for the lining to withstand internal water pressure. The data indicate that the internal lining (typically cement mortar) does in some cases, prevent corroded DI pipes from leaking.

Value of Standard Asphaltic Coating

A 1 mil thick asphaltic shopcoat is typically applied to DI pipe during manufacturing to prevent surface corrosion during transport. A recent report[14] indicates that the asphaltic shopcoat provides some limited corrosion protection underground. The results of the analyses discussed in this paper do not agree with that conclusion.

According to the evaluations included in this analysis, the number of failures of shopcoated DI and the age of the pipe at the time of the failures provide no indication that the 1 mil asphaltic shopcoat has any value with respect to corrosion protection underground. Furthermore, corrosion on DI has been documented[9] directly under undamaged asphaltic coating.

Conclusions Resulting from Analyses of 60 DI Pipe Evaluations

* The 10-Point system[12] does not appear to provide reliable data regarding the overall corrosivity of site soils nor does it application correlate well with the actual condition of DI pipe.
* DI piping corrodes at locations where PE wrap is damaged and pipe is exposed to soil.
* Corrosion can occur under intact PE encasement if water and/or soil migrate between the wrap and the pipe.
* PE encasement is not adequate for corrosion control of DI pipe in corrosive soil if the risk of pipe failure is not acceptable.
* The internal cement mortar lining provided with most DI piping can bridge across small penetrations in DI piping and prevent leakage for some time.
* The standard factory applied asphaltic coating that is applied to DI pipe provides no meaningful protection from external corrosion.

References

1. F.E. Stetler, "Accelerating Leak Rate in Ductile Cast Iron Water Mains Yields to Cathodic Protection", MATERIALS PERFORMANCE, Vol. 19, No. 10, October 1980, NACE International, Houston, TX.
2. R.A. Gummow, "The Corrosion of Municipal Iron Watermains", MATERIALS PERFORMANCE, Vol. 32, No. 3, March 1983, NACE International, Houston, TX.
3. J.A. Jakobs and F.W. Hewes, "Underground Corrosion of Water Pipes in Calgary, Canada", MATERIALS PERFORMANCE, Vol. 26, No. 5, May 1987, NACE International, Houston, TX.
4. B.J. Doherty, "Controlling Ductile-Iron Water Main Corrosion", MATERIALS PERFORMANCE, Vol. 29, No. 1, January 1990, NACE International, Houston, TX.
5. M.J. Szeliga, "Corrosion Failures in the Water Industry", AWWA Chesapeake Annual Conference, August 1990, American Water Works Association, Denver, CO.

6. R.A. Gummow, "CP Corrosion Control of Municipal Infrastructure", MATERIALS PERFORMANCE, Vol. 39, No. 2, February 2000, NACE International, Houston, TX.
7. M.J. Szeliga and D.M. Simpson, "Corrosion of Ductile Iron Pipe: Case Histories", MATERIALS PERFORMANCE, Vol. 40, No. 7, July 2001, NACE International, Houston, TX.
8. B. Spickelmire, "Corrosion Considerations of Ductile Iron Pipe-A Consultant's Perspective", Western Area Corrosion and Educational Conference, Tigard, Oregon, October 2001, NACE International, Houston, TX.
9. M.J. Szeliga and D.M. Simpson, "Evaluating Ductile Iron Pipe Corrosion", MATERIALS PERFORMANCE, Vol. 42, No. 7, July 2003, NACE International, Houston, TX.
10. B. Spickelmire, "Ductile Iron Corrosion Factors to Consider and Why", ASCE International Conference on Pipeline Engineering and Construction, Baltimore, Maryland, July 2003, ASCE, Reston, VA.
11. M.J. Szeliga, "Ductile Iron Pipeline Failures", MATERIALS PERFORMANCE, Vol. 44, No. 5, May 2005, NACE International, Houston, TX.
12. ANSI/ASTM C105/A21.5, "Polyethylene Encasement for Ductile-Iron Pipe Systems" (New York, NY: ANSI and Denver, CO: AWWA).
13. R.G. Wakelin and R.A Gummow, "A Summary of the Findings of Recent Watermain Corrosion Studies in Ontario", The International Symposium on Materials Performance Maintenance, Ottawa, Ontario, Canada, August 1991, The Metallurgical Society of CIM.
14. D.H. Kroon, D. Lindemuth, S. Sampson, and T. Vincenzo, "Corrosion Protection of Ductile Iron Pipe", Corrosion 2004, paper no. 4046, NACE International, Houston, TX.

FIGURE 1
DIP Corrosion Data Summary

Case Number	Location Region	Type of Main	Diameter (inches)	Polyethylene Encased	Age (years)	Original Thickness (inches)	Pit Depth (inches)	Average Corrosion Rate (mils/year)	Lining Delayed Failure	10-Point System Value
1	SE USA	water	16	yes	5	0.34	penetrated	68.000	nd	nd
18	SW USA	water	24	yes	10	0.33	penetrated	33.000	nd	nd
7	NE USA	sewer	8	no	16	0.36	penetrated	22.500	nd	5
8	NE USA	sewer	8	no	16	0.36	penetrated	22.500	nd	5
9	NE USA	sewer	8	no	16	0.36	penetrated	22.500	nd	5
10	NE USA	sewer	8	no	16	0.36	penetrated	22.500	nd	11.5
23	E Canada	water	6	no	15	0.32	penetrated	21.333	nd	6
28	E Canada	water	12	no	15	0.31	penetrated	20.667	yes	14.5
32	SE USA	water	6	no	16	0.31	penetrated	19.375	nd	2
24	E Canada	water	14	no	19	0.33	penetrated	17.368	yes	16
2	SE USA	water	16	yes	20	0.34	penetrated	17.000	nd	23.5
3	NE USA	sewer	12	no	22	0.37	penetrated	16.818	yes	14.5
25	E Canada	water	6	no	15	0.25	penetrated	16.667	nd	20
36	E Canada	water	12	no	24	0.40	penetrated	16.667	yes	16
41	SE USA	water	4	no	20	0.31	penetrated	15.500	yes	1
16	SE USA	water	8	no	20	0.30	penetrated	15.000	yes	23.5
17	SE USA	water	8	no	20	0.30	penetrated	15.000	yes	23.5
26	E Canada	water	6	no	22	0.33	penetrated	15.000	yes	9
29	E Canada	water	6	no	22	0.33	penetrated	15.000	nd	14.5
31	NE USA	sewer	6	yes	19	0.28	penetrated	14.737	nd	21.5
27	E Canada	water	8	no	21	0.30	penetrated	14.286	yes	12.5
20	NW USA	nd	16	yes	13	0.34	0.18	13.846	na	nd
50	NE USA	water	16	no	23	0.43	0.318	13.826	na	11
13	NE USA	water	16	no	26	0.43	0.340	13.077	na	12
34	NE USA	water	12	no	7	0.37	0.09	12.857	na	9
6	NE USA	sewer	8	no	16	0.36	0.19	11.875	na	5
49	SE USA	water	16	yes	20	0.34	0.23	11.500	na	14
51	NE USA	water	16	no	23	0.43	0.263	11.435	na	14
30	E Canada	water	8	no	27	0.30	penetrated	11.111	yes	16
60	SE USA	water	12	no	35	0.43	0.385	11.000	nd	3
37	E Canada	water	6	no	23	0.25	penetrated	10.870	yes	9.5
44	NE USA	sewer	12	no	31	0.31	penetrated	10.000	no	15.5
52	NE USA	sewer	12	no	31	0.31	penetrated	10.000	no	15.5
53	NE USA	sewer	12	no	31	0.31	penetrated	10.000	no	15.5
56	SE USA	water	12	no	35	0.43	0.343	9.800	nd	9.5
5	NE USA	sewer	12	no	23	0.37	0.215	9.348	na	8
11	NE USA	water	16	no	26	0.43	0.24	9.231	na	2
12	NE USA	water	16	no	26	0.43	0.24	9.231	na	1
42	SE USA	water	12	no	35	0.43	0.32	9.143	na	14
55	SE USA	water	12	no	35	0.43	0.301	8.600	nd	14
19B	SW USA	water	6	yes	8.75	nd	0.068	7.771	na	nd
19A	SW USA	water	6	yes	8.75	nd	0.043	4.914	na	nd
57	NE USA	effluent	18	no	35	0.41	0.27	7.714	na	8
4	NE USA	sewer	12	no	23	0.37	0.16	6.957	na	1
45	NE USA	sewer	8	no	29	0.30	0.20	6.897	no	2
35	NE USA	water	12	no	22	0.31	0.15	6.818	na	1
47	NE USA	sewer	16	no	22	0.34	0.14	6.364	na	8
58	NE USA	effluent	18	no	35	0.41	0.22	6.143	na	10
38	NE USA	sewer	8	yes	19	0.30	0.11	5.789	na	7
43	SE USA	water	12	no	35	0.43	0.20	5.714	na	12.5
33	SE USA	water	16	no	30	0.40	0.16	5.333	na	6
59	NE USA	effluent	18	no	35	0.41	0.17	4.857	na	0
46	NE USA	sewer	16	no	22	0.34	0.10	4.545	na	11
54	SE USA	water	16	yes	20	0.34	0.08	4.000	na	3
48	NE USA	sewer	16	no	22	0.34	0.07	3.182	na	1
39	SE USA	water	16	yes	20	0.34	0.06	3.000	na	5
14	SW USA	water	8	no	5	nd	penetrated	nd	nd	nd
15	SW USA	water	8	no	17	nd	penetrated	nd	nd	nd
21	SW USA	sewer	10	yes	16	nd	penetrated	nd	yes	nd
22	W Canada	water	nd	yes	15	nd	penetrated	nd	yes	nd
40	NW USA	water	8	yes	12	nd	penetrated	nd	nd	nd

Notes: 1. 19A at tear in polyethylene encasement, 19B under intact polyethylene encasement.
2. nd =no data
3. na = not applicable

LADAR-based Pipeline Inspection and Location

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Abstract

Laser Radar (LADAR) enables a new generation of quantitative pipeline inspection techniques. These techniques differ from traditional inspection techniques including Closed Circuit Television (CCTV) and laser profiling systems by capturing 3-D range images that encode the local three-dimensional (3D) structure of the inspected pipeline. Individual range images thus obtained can be processed and integrated into a high-resolution geometric model of the inspected pipeline. Once the integrated pipeline model is assembled, techniques from 3-D computer vision and computational perception produce quantitative answers to pipe location, integrity, and performance questions. Three separate case studies that employed LADAR in quantitative pipeline inspection operations are presented: estimation of the effects of relining on pipe transport capacity within a culvert system, estimation of corrosion in Pre Cast Concrete Pipes (PCCP) and geo-location of two parallel segments of brick-lined sewer. In each case, an overview of the approach to data acquisition, processing, and sample results are provided.

I. Introduction

A periodic program of sewer inspections is necessary to determine pipe condition and prioritize maintenance to preserve integrity of public utilities (EPA, 1999). Leaks, clogs or collapses can disrupt services and in the most severe cases result in significant damage to the environment and economy of affected communities. The most common forms of sewer inspections employ trained operators armed with CCTV cameras in order to identify and grade pipe defects. However such inspections rely on inspection vigilance and do not produce a quantitative description of pipe condition. Furthermore, image-based inspection systems, such as CCTV, cannot produce rigorous cross-sectional analysis, three-dimensional representation of pipe features, or pipe location information.

In this paper, we present a unique combination of components for quantitative pipeline inspection: a robust mobile platform, called Responder (Figure 1), a sensor suite that includes a 3-D laser scanner, and 3-D data processing techniques derived from computer vision and computational perception (Horn, 1986) (Russell, 1995). The geometric integration and processing of 3-D point clouds rely on state-of-the-art techniques (O'Neill, 1997) (Hartley, 2000). In addition, using Redzone analytics, 3-D geometric models of pipes can be reconstructed and used to produce quantitative answers to pipe location, integrity, and performance questions.

Figure 1: The Responder Robotic Platform

Figure 2: Deployment of Responder

Three separate case studies that employed LADAR in quantitative pipeline inspection operations are presented. In the first case study, the LADAR was employed to estimate the effects of relining on pipe transport capacity within a culvert system. The second case study details the use of LADAR sensing in the precision estimation of corrosion in PCCP pipes. The last case examines the use of LADAR in the geo-location of two parallel segments of brick-lined sewers. In each case, the basic approach to data acquisition, processing, analysis, and project outcomes is presented.

The rest of this paper is organized as follows: Section II presents the Robotic platform developed, Section III, IV, and V detail the three different applications mentioned above. Section VI concludes this paper.

II. The Responder Robotic Platform

RedZone's Responder robot platform (Figure 1) features a 4 HP hydraulic drive motor, low-profile tracks, and skid-steering with independent track control. It can produce over 35 KN of force to pull its steel armored umbilical cables from a semi-automated control reel. It weighs 270 kg and has a volume profile of 58 cm W x 96 cm L x 51 cm H. It can be equipped with various work attachments such as sediment resuspension, sampling, and emergency response tools. The platform's maximum operating depth is 150 meters and can survey nearly 7 kilometers of pipes from a single access point with a maximum speed of 12 m/min in its standard configuration.

For comprehensive pipe inspections Responder can be equipped with a sensor suite which includes a multi-frequency sonar, PTZ CCTV camera, high-resolution imager, multi-gas logger, as well as a 3-D laser scanner. Sensors can be mounted on a turret capable of 360° motion range. Additional sensors can be easily integrated via Responder's Electronics Box (E-Box) which provides a range of expansion ports.

Responder is designed to be deployed from a standard manhole (Figure 2). It can operate with a full suite of inspection sensors in pipes 92 cm and above, either charged, partially charged or dry.

Responder is outfitted with a three dimensional laser scanning system. This system was used in each of the three subject case studies. A scan-line laser is centrally rotated, producing a hemispheric scan as illustrated in Figure 3. This means of scanning is advantaged over 2-D laser-profiling systems where a single cross section is obtained. The significance of the advantage is detailed in the case studies.

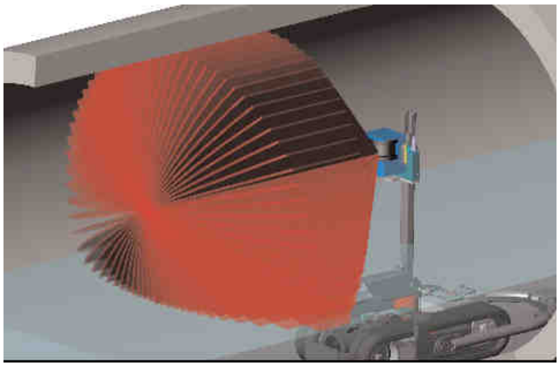


Figure 3: Laser scanner deployed

The first case presents the advantages of 3-D laser scanning when applied to pipe cross-sectional analysis problems.

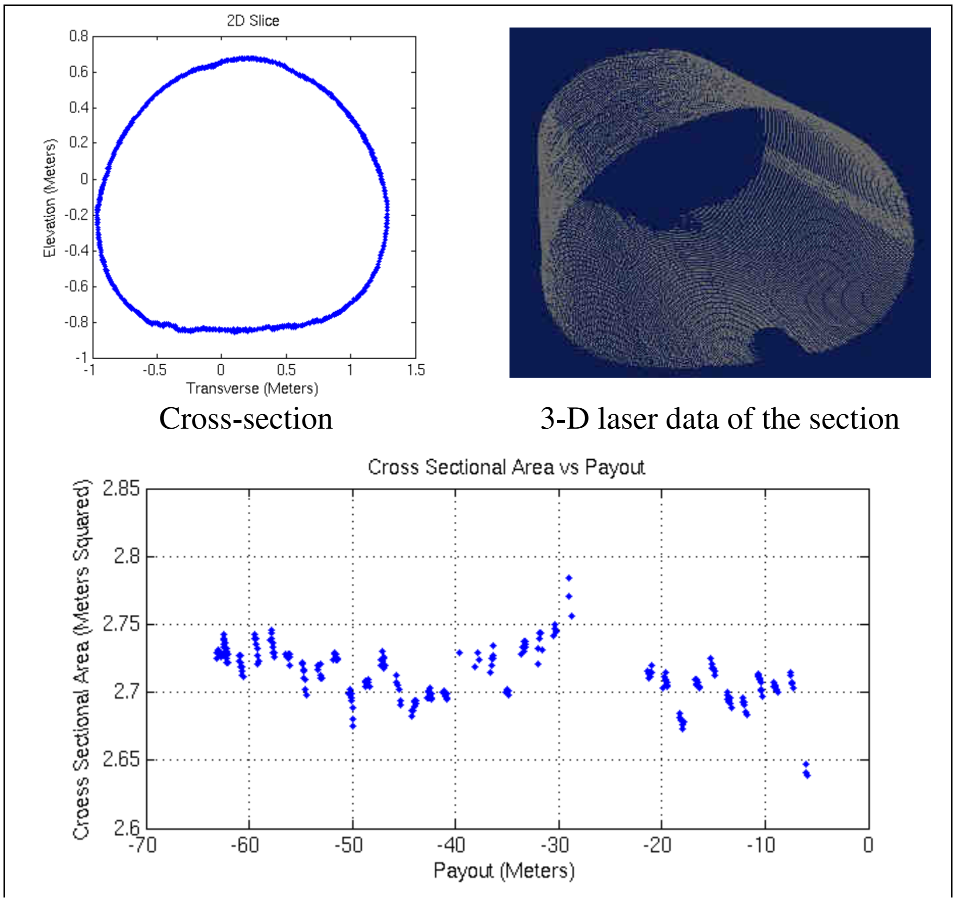
III. Application to the study of lining on pipe capacity

As mentioned in section II, three dimensional laser scanning is able to efficiently and inexpensively provide precision cross sectional area and perimeter measurements within a broad range of pipe shapes and sizes. This first case study outlines a laser-based inspection of a culvert to estimate the effects of relining on transport capacity. The approach to computing transport capacity utilized the Manning Equation (Chaudhry, 1993) to calculate the open-channel flows. As can be seen in Equation 1, flow calculations are highly sensitive to the accuracy of area, A, and wetted perimeter, P. Customer specification required one sigma area and perimeter with mean error of no more than 1.5%.

$$\text{(Eq. 1)} \quad Q = A\frac{k}{n}\left(\frac{A}{P}\right)^{\frac{2}{3}} S^{\frac{1}{2}}$$

The relined culvert presented an egg-shaped, elliptical cross section (approximately 1.5 x 2.3 meters) and was nearly 60 meters in length. Accurate calculation of transport capacity required obtaining area and perimeter estimates in 0.3 meter increments over the entire 60 meter expanse. Responder was outfitted with the scanning laser (approximately centered) and deployed to acquire the required 200 precision 3-D laser scans (one scan for each 0.3 m increment). From each individual scan, a composite perimeter and area estimate was synthesized after pipe axis computation, laser data realignment, and correction for occlusion of the pipe invert[1].

An example scan collected by Responder is shown in Figure 4 (top right). Two-dimensional cross sections, such as the one from Figure 4 (top left) were extracted from calibrated and registered 3D scans. Due to the uncertainty associated with fabrication and liner installation and the variable cross-section, accurate as-built perimeter and area function was not known. However, standard convex hulling algorithms were applied to the cross sections to calculate area and perimeter. The collected cross section areas (Figure 4, middle) and perimeters (Figure 4, bottom) were delivered to contractors for analysis.



[1] There was a small amount of water in the pipe and requiring manual measurements the water column at various locations within each scan.

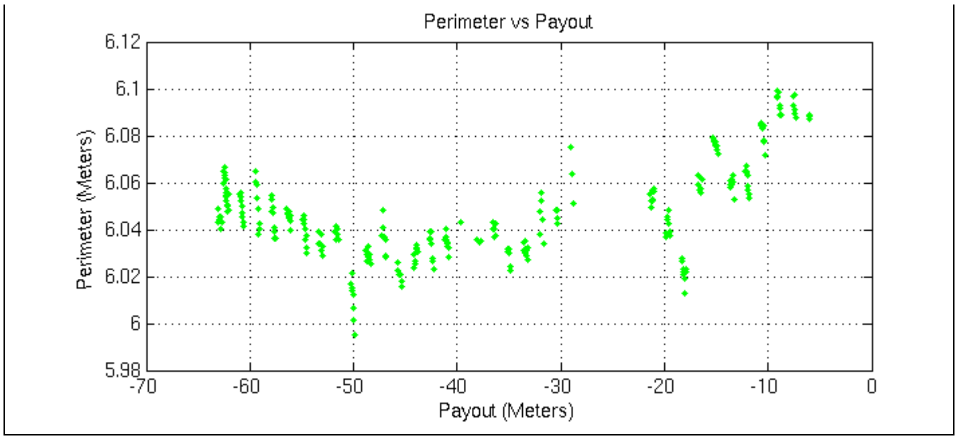


Figure 4: Results on the lining influence on the pipe capacity.

While the results of the hydraulic analysis are proprietary and outside the purview of this paper, the accuracy of the areas and perimeter estimation techniques were verified on pipes of known area and perimeter. As shown in Table 1, the estimation techniques easily satisfy the 1.5%, single sigma average error requirement.

Scan Attempt	Expected Area (Meters Squared)	Mean Area (Meters Squared)	Standard Deviation
1	1.5077	1.5069	0.0003
2	1.5077	1.5050	0.0004
3	1.5077	1.4965	0.0008
4	1.5077	1.5049	0.0007
5	1.5077	1.5040	0.0007
6	1.5077	1.5021	0.0007

Table 1: Measured areas for test pipe.

The presented case study provides superior results in cross-sectional analysis (area, perimeter, ovality, eccentricity, etc.) over those possible with 2-D laser profilers. The primary reason for this improvement is that 2-D profilers implicitly assume that the profile is perpendicular to the central axis of the pipe. If this constraint is violated, i.e. the perpendicularity is off by even a few degrees, the cross-section is corrupted. Three-dimensional scans are immune to this problem, since cross-sections that are perpendicular to the pipe axis can always be obtained. The superior priorities of 3-D laser data are exploited in the next section in a pipe corrosion application.

IV. Application To Pipe Corrosion Evaluation

The second case overviews the use of 3-D laser scanning in the measurement of pipe material loss (corrosion) and gain (buildup). CCTV and other image-based inspection technologies provide subjective, visual estimations of material loss or gain (see Figure 5 top). As an example, estimates of material loss have been made from CCTV images of exposed rebar by estimating the rebar depth from the as-built drawings and inferring material loss accordingly. However, such cases are rare and CCTV images are ambiguous with respect to dimension. In particular, CCTV images do not provide the means to make quantitative measurements of either corrosion or buildup.

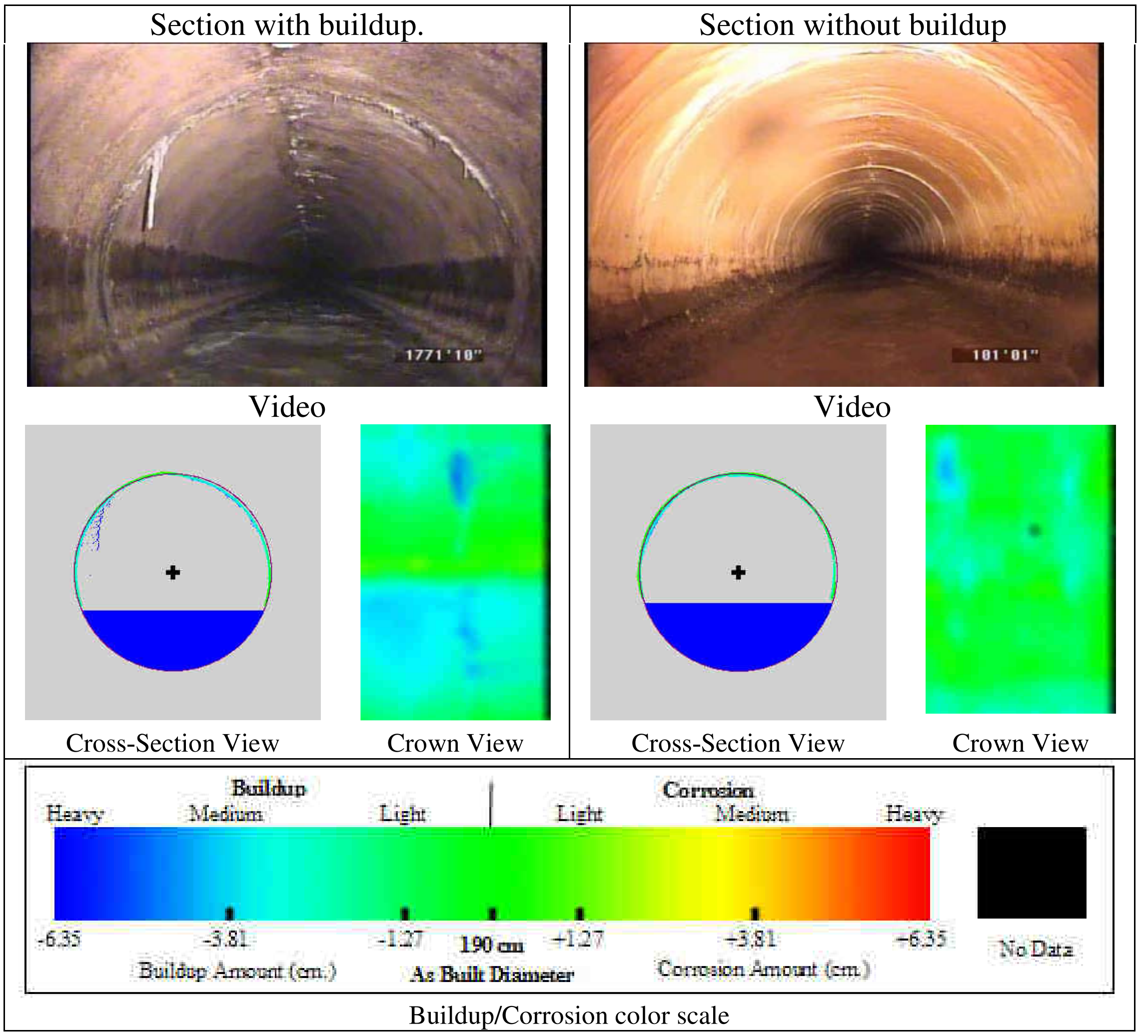


Figure 5: Pipe corrosion evaluation results.

Responder's 3-D scanning LADAR system was employed within a segment of concrete pipe that was 1.9 meters in diameter and over 790 meters in length. Within this segment, 3-D snapshots where acquired at approximately 5 meter intervals in order to provide full inspection coverage. Post acquisition analytics operate on acquired laser data to compute the pipe axis, correct for sensor alignment issues, and derive pipe coordinates frames. After alignment, additional analytics convert calibrated data in to quantitative cross-section (Figure 5, middle left) and crown corrosion (Figure 5, middle right) views.

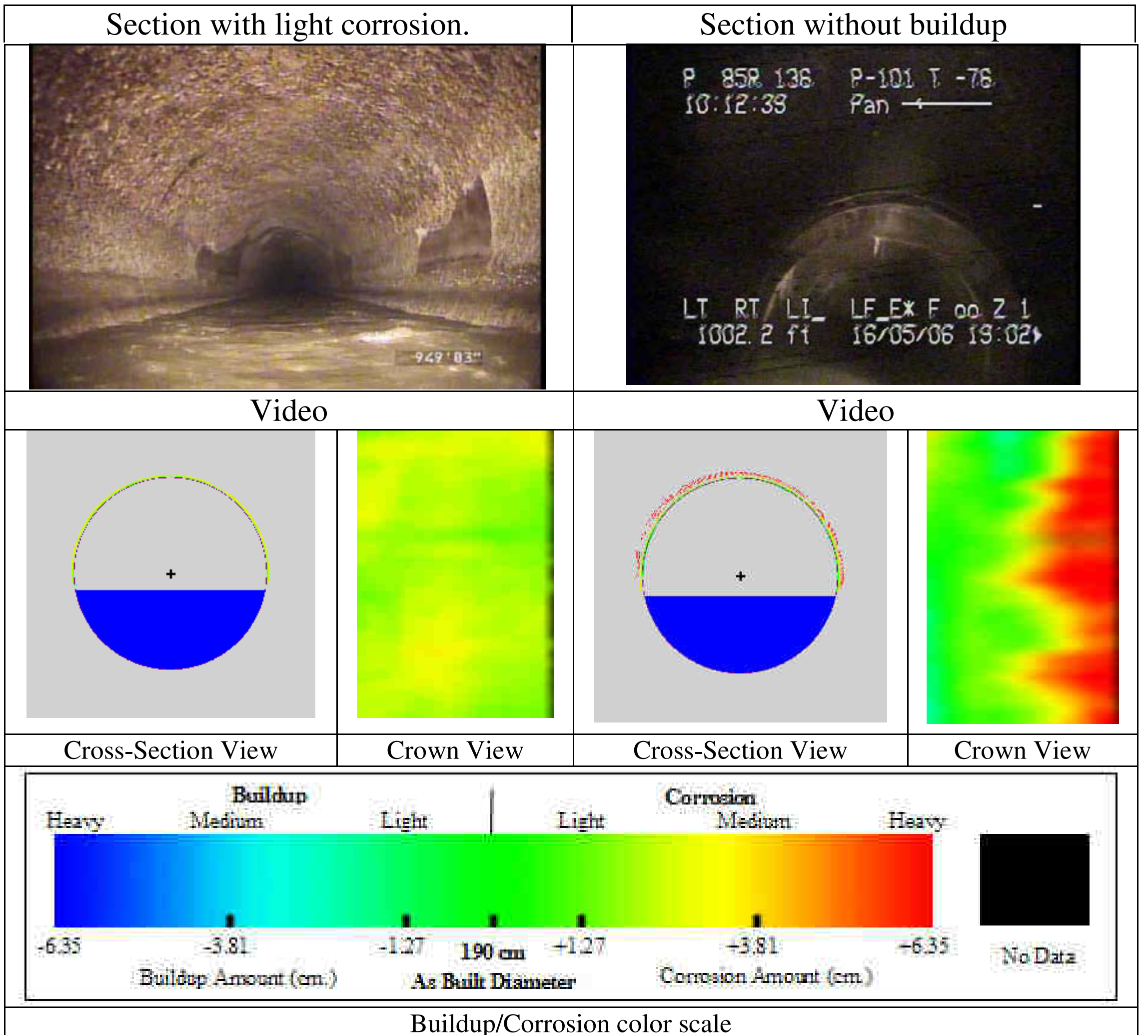


Figure 6: Corroded Pipe Example

Due to the length of pipe inspected, it was important to automatically and visually highlight areas of interest and concern. Quantitative corrosion and build up maps are colored (Figure 6, bottom) to highlight deviations from as-built or expected pipe conditions. Yellow, Orange, and Red coloring indicates that the wall is increasingly further away from the center of the pipe than expected - highlighting material loss or corrosion. Aqua and Blue coloring indicates that the wall is increasingly closer to the center of the pipe than expected - highlighting material gain or buildup. Green coloring indicates that the wall is at an expected distance from the center of the pipe. For example, the root intrusion shown in Figure 5 is easily identified from the CCTV images and easily measured using the cross-section maps.

In Figure 6 (middle, left) there is an approximately uniform loss of concrete on each of clocking angles. Loss of pipe material is noticeable in the video screen capture (Figure 6, top left) but the amount of pipe loss is nearly impossible to estimate with CCTV imagery. Nevertheless, material loss is accurately measured with the cross-section view (Figure 6, bottom left) and crown view corrosion (Figure 6, bottom right) maps.

Another provided representation of material loss/gain is as a graph of average pipe diameter versus relative location within the pipe (Figure 7). This graph highlights areas of advanced corrosion and buildup that warrant further inspection in cross-section or crown views.

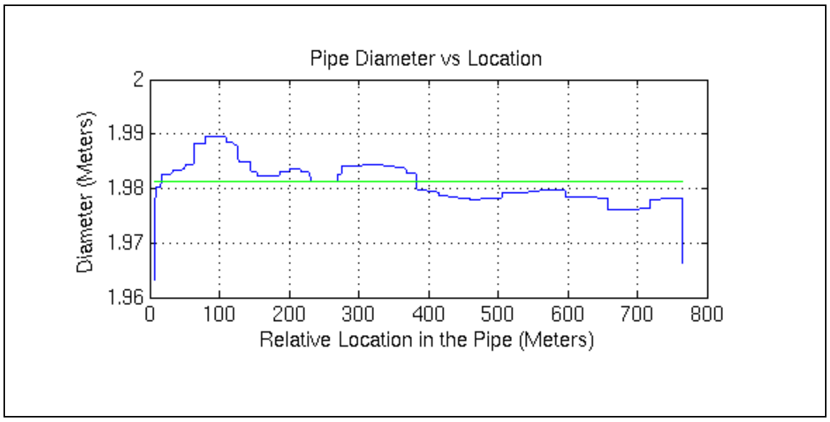


Figure 7: Example Average Pipe Diameter versus Relative Location

The second case provided a brief overview of the use of 3-D Laser scanning in the estimation of pipe corrosion and buildups. Similar to the first case study, 3-D laser scanning provides superior results due to the elimination of sensor alignment problems. The third and final exploration of laser technology in the paper applies 3-D laser data to a pipe geo-location problem.

V. Application to Geo -location of Parallel Pipes

The penultimate use of LADAR technology is in the geo-location of buried infrastructure. The final case study presents the location of parallel sewer pipes near a construction site in order to prevent accidental damage during excavation. The pipes ran adjacent and beneath a system of railroad track and complex of abandoned surface structures. Various design drawings and surveying documents (see Figure 8) were available but they were inconsistent, dated (late 1800's), known to be inaccurate, and insufficient for planning site excavation and construction to avoid damaging the sewer system.

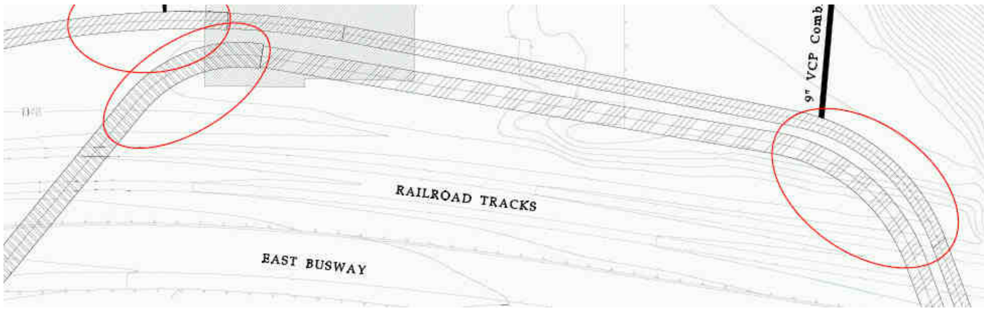


Figure 8: Expected position of parallel Belgian block storm sewer lines.

Pipe construction was Belgian blocks with horseshoe cross section of heights 2.7 and 1.8 meter and extended 345 m and 123 m in length, respectively. While the segments were parallel for a portion of the plan view, they were composed of three approximately piece-wise linear segments separated by two elbows - highlighted in red within Figure 8.

Each pipe segment was delimited by two manholes that provided GPS coordinates, survey references, and deployment access to Responder. Three-dimensional laser snapshots (see Figures 9 and 10) were acquired at regular intervals to compliment the continuous acquisition of vehicle position data.

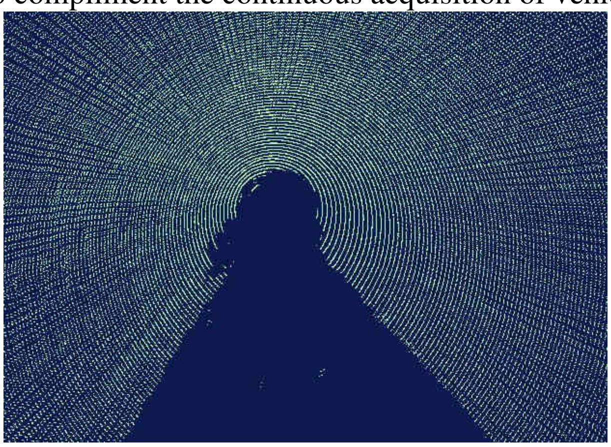


Figure 9: Example of featureless laser scan

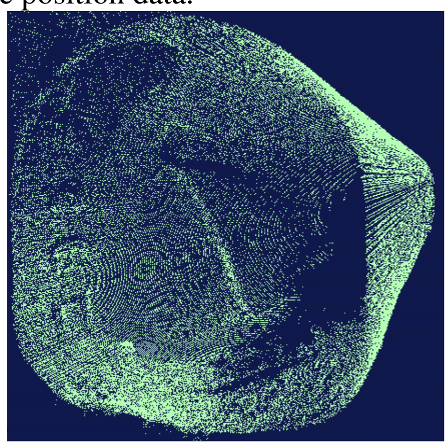


Figure 10: Example of laser scan near a man hole

Accurate geo-location for excavation planning required the fusion of multiple position estimation techniques in order to meet the location constraints (approximately one pipe diameter) data over the length of the inspection. The first technique relied on standard "dead-reckoning techniques" (Chung, 2001). Dead-reckoning relies on the use of orientation (roll, pitch, and yaw) and odometer information (umbilical length) to reconstruct the position of the pipe. This method is know to accumulate error and is particularly vulnerable at bend locations where umbilical position is highly variable.

The second approach relies on the co registration of three-dimensional LADAR snapshots from consecutive scans using the Iterative Closest Point (ICP) algorithm (Rusinkiewicz, 2001). This approach is also known to accumulate errors but is particularly sensitive to the lack of features in the linear section of the pipe, as shown in Figure 9. However, it proves accurate in feature rich areas of the pipe such as man-hole locations (Figure 10), pipe junctions, and at the elbows.

Position estimates were derived from a fusion of both approaches and the location of the manholes to estimate the pipe locations as presented in Figure 11. Careful digging was undertaken at various locations and confirmed the pipe position with enough accuracy to avoid damage during construction as intended.

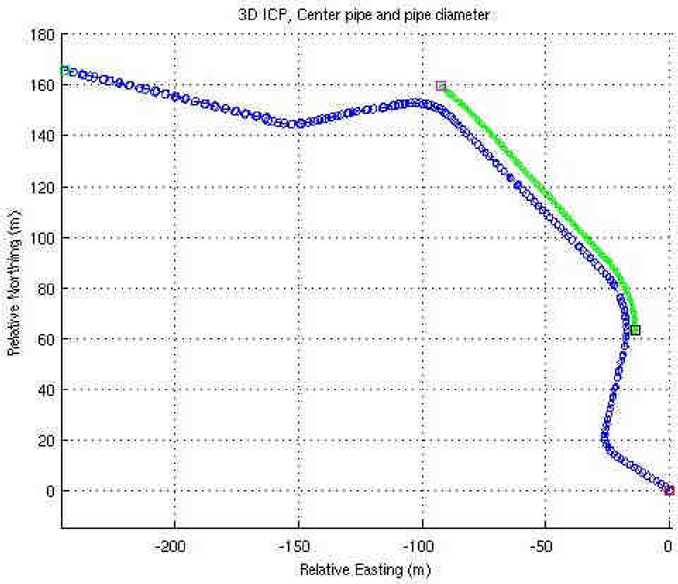


Figure 11: Pipes estimated location

In this section we presented the use of Responder for the geo-location of two parallel pipes. The use of point cloud registration techniques allowed us to produce an accurate 3D reconstruction over hundreds of meters of pipe.

VI. Conclusion

In this paper, we presented a pipeline inspection system composed of three unique components: a robust mobile platform, a 3-D LADAR scanner, and innovative data processing techniques. We also presented three case studies that highlight the unique quality of 3-D LADAR to produce quantifiable pipe inspections with superior accuracy than is possible with 2-D profiling lasers. The first case study employed LADAR to recover cross-sectional areas and wetted perimeter estimates to compute open flow pipe transport capacity. The second utilized LADAR to estimate corrosion and buildup in PCCP pipe to prioritize maintenance activities within the inspected line. In the final study, LADAR pipe position estimates were fused with standard “dead” reckoning techniques to guide site excavations on a construction site. In each case, 3-D LADAR provides the precision geometric and structural data that is complementary to standard inspection methods, such as CCTV, and offers a new suite of precision diagnostic tools for pipe inspection and maintenance operations.

VII. REFERENCES

(EPA, 1999) Environmental Protection Agency, Collection Systems O & M Fact Sheet, EPA 832-F-99-031, September 1999

(Chaudhry, 1993) Chaudhry, M. H. Open-Channel Flow. Prentice-Hall, Inc., 1993.

(Lightman, 2006] Lightman, A. (2006). “Sewer Assessment in Fort Worth, Texas”. Trenchless Technology, January.

(Rusinkiewicz , 2001) S. Rusinkiewicz and M. Levoy. Efficient Variants of the ICP Algorithm. Third International Conference on 3D Digital Imaging and Modeling, 2001.

(Chung, 2001) H. Chung, L. Ojeda, and J. Borenstein, “Sensor fusion for Mobile Robot Dead-reckoning with a Precisoion-calibration Fiber Optic Gyroscope” In IEEE ICRA, pages 3588-393, Seoul, Korea, May 21-26, 2001.

(Horn, 1986) B. K. P. Horn, Robot Vision, MIT Press, Cambride, Massachusetts, 1986.

(Russell, 1995) S. Russel and P. Norvig, Artificial Intelligence: A Modern Approach, Prentice Hall, New Jersey, 1995.

(O’Neill, 1997) B. O’Neill, Elementary Differential Geometry, Academic Press, San Diego, CA, 1997.

(Hartley, 2000) R. Hartley and A. Zisserman, Multi-View Geometry in Computer Vision, Cambridge University Press, Cambridge, UK, 2000.

Condition Assessment and Rehabilitation Recommendations to Renew Raw Water Pipeline Infrastructure for City of Atlanta

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Abstract

The City of Atlanta, like many other cities, is currently facing the challenge of rehabilitation or replacement of much of its existing buried infrastructure. To confront this challenge, Mayor Shirley Franklin has led Atlanta on an aggressive system-wide capital improvement program. In order to make the most efficient use of available funds for pipeline renewal, the City's Watershed Management Department has committed to explore innovative and cost-effective options, including trenchless technologies. A significant step forward in this program has been a condition assessment and rehabilitation analysis for the system-critical raw water pipelines that connect the City's water source, the Chattahoochee River, to the Hemphill Reservoirs and Hemphill and Chattahoochee Water Treatment Plants (WTPs). The objective of this paper is to discuss the methods and results of this condition assessment and rehabilitation analysis project.

The raw water mains servicing the Hemphill and Chattahoochee WTPs consist of three cast iron pipelines laid in 1893 (30-inch), 1907 (36-inch), and 1924 (48-inch), and a 72-inch welded steel pipeline (WSP) laid in 1973 (bell and spigot welded lap joints). The approximate lengths of these pipes are: cast iron (54,000 feet total) and steel (25,000 feet).

Key elements of the condition assessment for all four pipelines included a leak detection effort, a soil corrosivity and pipe corrosion evaluation, and a structural integrity assessment of the pipelines. Additional components of the 72-inch WSP assessment included an evaluation of existing corrosion protection measures and a detailed investigation of pipeline structural integrity with an emphasis on prior failure mechanisms. Testing and destructive evaluation of samples cut from the pipeline were critical to the assessment; however, balancing high water demands and the risk of catastrophic failure during removal of the samples required an innovative and highly coordinated effort.

A rehabilitation analysis of the four pipelines followed the condition assessment. The analysis included consultation with multiple pipe and materials vendors and involved the use of hydraulic calculations to evaluate conveyance capacity implications of the various rehabilitation alternatives. The analysis also included the goal of extending the life of these assets by 100 years. The final report indicates that portions of these large existing mains are indeed candidates for rehabilitation.

Background

In order to supply two primary WTPs, the City of Atlanta is permitted to withdraw 180 million gallons per day from the Chattahoochee River. Raw water is gravity fed from an intake structure on the river to a raw water pumping station and then the water is pumped to the Chattahoochee WTP and the Hemphill WTP. While the Chattahoochee WTP lies within 1,600 linear feet (LF) of the pumping station, the Hemphill WTP is 3.4 miles away via the cast iron mains, 5 miles away via the steel main, and 230 feet higher in elevation.

In March 2005, the City of Atlanta contracted with the CH2M HILL/Williams-Russell & Johnson, Joint Venture (JV) to evaluate the condition of these raw water transmission pipelines and the feasibility of rehabilitation to extend the service life of this critical infrastructure.

The configuration of the raw water transmission system to the Hemphill WTP was defined in the 1890s during the initial construction of the Hemphill WTP, the first raw water storage reservoir, and the raw water pumping station. The pipeline characteristics are summarized in Table 1.

These critical pipelines have experienced a number of leak and failure incidents over their history. Considering the age of the cast iron pipe and a prior suspicion of material problems with the steel pipe, there was pressure to begin a design for total replacement with new pipelines. However, in view of the significant cost and community impact of completely new construction, the Atlanta engineering staff decided to assess these mains as candidates for a trenchless rehabilitation technology.

Diameter	Length	Material	Joint Type	In-Service Date
30-inch	~18,000 LF	Cast Iron (Class B)	Bell and Spigot	1893
36-inch	~18,000 LF	Cast Iron (Class B)	Bell and Spigot	1908
48-inch	~18,000 LF	Cast Iron (AWWA 7C.1-1908, B&C)	Bell and Spigot	1924
72-inch	~24,300 LF	Steel (AWWA C202-64-B)	Welded Lap Joint	1975

Table 1. Summary of existing raw water pipelines to Hemphill WTP.

Condition Assessment Objective

The project's scope of work included two primary objectives: (1) to assess the condition of the raw water transmission system and (2) to develop feasible rehabilitation alternatives that would provide long-term solutions for raw water conveyance. The City of Atlanta has set a goal of 100 years for the service life of new and re-newed buried infrastructure.

Condition Assessment Approach

The process of developing a detailed assessment plan for the four raw water pipelines featured a threat-based approach. While pipeline rehabilitation is a common practice in water distribution systems and large sewer lines, the rehabilitation of large-diameter transmission pipelines is less common. The critical function of large transmission lines creates a risk for utilities to take the lines out of service, limiting internal access to the lines for assessment and renovation. The shortage of information on previous projects at this scale (with this mix of pipe materials and conditions) created the need for specific analysis, since no set of standard solutions could be applied. A custom condition-assessment scope was developed to address the specific threats to the long-term integrity of these pipelines that had been reported by City of Atlanta staff.

The approach for the three cast iron raw water pipelines was focused more toward corrosion assessment than structural analysis. The history of the three cast iron pipelines suggested the pipelines were brittle and assembled with fragile lead solder and oakum joints. There were no reports of unique structural problems or systematic metallurgical defects, as was reported for the 72-inch steel pipeline. The cast iron pipelines were primarily assessed for soil corrosivity along the pipeline alignment, the state of corrosion of the cast iron, and leakage. Concurrently with the project, a leakage investigation was performed by another City of Atlanta team which at the time reported no leaks in the system.

The assessment scope for the 72-inch steel pipeline was developed after a thorough review of available historical condition assessment and failure history documents. The 72-inch steel pipeline was reported to be lacking an effective cathodic protection system and the pipeline had a history of structural failure at the joints. Therefore, the scope of the 72-inch steel pipeline assessment was tailored to address combined threats of corrosion and structural weakness; however, emphasis was placed on the structural investigation.

Phase 1 – Scope Summary

Phase 1 focused on indirect assessment techniques, including soils evaluations and other assessments that could be performed from the ground surface without the support of excavation equipment. Some hand excavation and use of a soil core machine were included in the indirect assessment, but in most cases "indirect" assessment implies "no-dig" activities and no confined space entry.

Phase 1 included the following tasks:

- Preliminary pipeline alignment and appurtenance survey (all pipelines)
- Pipeline leakage and void detection (all pipelines)
- Above-ground geological/geotechnical assessment (all pipelines)
- Soil corrosivity testing at 1,000-foot intervals along cast iron pipelines
- Evaluation of existing corrosion prevention measures for 72-inch steel pipeline

Phase 2 – Scope Summary

Phase 2 included direct inspections of the pipeline exterior and interior surfaces, and cutting of pipe samples for laboratory analysis from two of the four pipelines. Phase 2 direct assessments required excavation and construction support, partial dewatering of the 72-inch steel pipeline, confined space entry at multiple locations along the pipeline, and insertion of a remote camera into the 72-inch steel pipeline. Direct assessment allowed a more "hands-on" evaluation of the infrastructure. The number of direct assessment sites was limited by: funding, a desire to minimize negative impact on the community with road closures, and a perceived risk of inducing failure(s) associated with the assessment activities. With this in mind, a minimal number of assessment sites representing diverse soil conditions were selected.

Phase 2 included the following tasks (cast iron mains):

- External assessment of the pipeline surfaces at four locations
- Soil corrosivity testing at each external inspection location
- Laboratory metallurgical and mechanical testing of a sample cut from the 48-inch cast iron pipe

Phase 2 included the following tasks (steel main):

- Excavation, isolation, and dewatering of the pipeline in preparation for Phase 2 assessments
- Geotechnical investigation of pipe bedding and backfill at four locations
- Structural assessment of external pipeline surfaces at four locations
- External corrosion and soil corrosivity assessment at four locations
- Structural and corrosion assessment of internal pipeline surfaces at three locations
- Internal inspection by remote video camera through 3,000 feet (12%) of the alignment
- Laboratory metallurgical and mechanical testing of a large sample cut from the 72-inch steel pipeline (included a complete welded bell and spigot lap joint for analysis)

- Laboratory analyses of internal cement mortar liner and external coal tar and felt coating

Assessment Implementation & Data Gathering

The following types of historical data were collected in support of the pipeline condition assessment: 1) Design and Construction History; 2) Previous Condition Assessments; 3) Pipeline Failure History; 4) Transmission System Operation; 5)Geophysical Data; and 6) Raw Water Quality Parameters.

Water chemistry can have detrimental effects on cast iron and steel pipelines, with aggressive water leaching cement from mortar linings and causing corrosion of exposed metal. Therefore, sources of historical water quality were identified; parameters of interest are reported in Table 2.

Parameter	Units	2006[a]	2005	2004	2003
Temperature	ºC	16.6	16.96	No Data	17.0
Total Alkalinity	mg/L	18.3	18.5	21.4	20.7
Hardness	mg/L as $CaCO_3$	20.6	22.6	21.9	32.2
pH	No Units	7.2	7.2	7.3	7.3
Dissolved Solids	mg/L	30.3	30.2	31.1	44.7
Langlier Index	No Units	-2.0	-1.9	-1.8	-1.7
Aggressive Index	No Units	9.8	9.8	9.9	10.1
CCPP[b]	No Units	-7.5	-7.6	-7.4	-6.9

[1]January to August 2006
[2]Caclium Carbonate Precipitation Potential

Table 2. Water quality data--summary of annual averages (river intake).

The objective of evaluating daily water and air temperature data was to confirm previous observations that failures of the 72-inch steel pipeline were related to cold water and air temperature. Reports associated with the 72-inch steel pipeline indicated that the three documented pipeline breaks (up to 1982) occurred during cold snaps when water temperature dropped below 40 degrees Fahrenheit.

From 1976 to 2006, a total of 30 weather periods were identified with minimum daily water temperatures below approximately 41°F. Three of the seven most recent major failure events on the raw water pipelines occurred when the water temperature was below 41°F (Jan. 15, 1982; Jan. 17-18, 1994; Jan./Feb. 2003). The fourth and fifth major pipe breaks occurred in December 1980 when the water temperature was approximately 42°F.

Pipeline metallurgy sampling, testing, and analysis

The structural investigation of the 72-inch steel main consisted of the following tasks:

1. Evaluate pipeline bell ends to determine if bells were "rolled" (formed with an offset belling die), a fabrication process that has been known to cause excessive cold working and embrittlement.
2. Evaluate the method of field joint assembly to determine if fit-up between bell and spigot was eccentric and/or fully stabbed. Eccentricity of the spigot within the bell can leave substantial gaps between the bell and spigot at the internal fillet weld and can be caused by over-sizing of the bell end during the manufacturing process. Fully stabbed joints result when the spigot has been placed as deeply as possible into the bell end, forcing the internal fillet weld to be located on the bell transition, or bell curve—a practice prohibited by versions of AWWA C-200 (Standard for Steel Water Pipe 6-Inches and Larger) and C-206 (Standard for Field Welding of Steel Water Pipe) since 1985.
3. Evaluate pipeline field weld quality and the presence of defects or material failures near the internal fillet welds.
4. Evaluate the integrity of factory spiral seam and coil splice welds for evidence of defects.
5. Evaluate the basic mechanical, chemical, and metallurgical properties of the base metal of the 72-inch steel pipeline barrel to identify any defects from fabrication and to determine if the metal conforms to the design specifications of AWWA C202-64T (Class B Tentative Standard for Mill-Type Steel Water Pipe).

To allow a structural investigation of a complete bell and spigot lap joint and the base metal of the pipeline barrel on both sides of the joint, a 4.5-foot sample section of the 72-inch steel pipeline was saw-cut and removed. The removed section was replaced with made-to-order spool manufactured by Northwest Pipe. A companion paper to this document, published in these proceedings (Jacob et al., 2007), provides a detailed discussion of the structural investigation of the 72-inch WSP.

Internal inspection of the pipeline indicated that all pipe joints investigated were fully stabbed, or nearly fully stabbed, and that the internal fillet weld was located on the curved bell transition. This field welding procedure is prohibited by revisions of AWWA C206 made since the pipe was originally installed. The reason for avoiding welding at this point on the pipe is to avoid increasing stresses in the most cold worked portion of the pipe.

Ultrasonic wall thickness measurements indicate there has been no significant erosion or corrosion of the steel material over its service life. Minimum wall thickness was observed to be only 0.021 inch less than the original nominal thickness of the pipe.

Structural investigation of the cast iron mains

A single 16-inch x 20-inch sample was cut from a 12-foot long pipe segment removed from an abandoned section of the 48-inch cast iron pipe alignment and submitted to a materials testing laboratory for the following analysis:

- Visual examination and precision measurements of sample surfaces to identify any coating, inclusions, pores, pits, wall loss, corrosion products, and other surface defects.
- Evaluation of tensile strength test results, longitudinal and transverse, to determine pipe wall material strength characteristics.
- Microstructure analysis of a prepared cross section cut from pipe sample to measure and classify types and sizes of graphite flakes and other metallurgical constituents.
- Chemical analysis to determine alloy chemistry.
- Brinell Hardness tests on interior and exterior surfaces of metal sample.

The results of the condition assessment of this sample indicated that the cast iron pipelines are at low risk of structural failure due to corrosion or other metallurgical factors. Additional samples were not taken due to the risk associated with cutting out sections or coupons, which would have resulted in leaving one of the cast iron mains out of service.

Soil Analysis

Based on site reconnaissance, it appears that larger scale geological factors did not contribute to past failures of the raw water transmission pipelines. Several indirect test methods were used to evaluate the potential for corrosion on buried steel and cast iron pipes. These tests included measurements of soil resistivity along the pipeline alignments, soil sample testing for corrosive constituents, and electrical tests to evaluate coating efficiency and cathodic protection (steel pipe).

Corrosion

For the cast iron pipelines, visual observations indicated that the exterior pipe wall sustained minimal corrosion at the locations where the pipes were exposed. The surface of the pipes exhibited some corrosion (rust), relatively shallow corrosion pitting (localized metal loss), and a thin layer of graphitization (residual carbon from which the iron has dissolved, or corroded, from the metal matrix). However, no significant metal loss or pitting was observed. To evaluate internal corrosion, sections of the 48-inch cast iron pipe were observed. A relatively thin cement mortar lining was present inside the pipes observed. The lining was intact but very weak (could be easily broken) throughout its thickness and it had poor adhesion to the pipe wall. The sample of the 48-inch cast iron pipe was analyzed to evaluate pipe metallurgy, chemistry, strength characteristics, and metal loss due to corrosion. Based on these tests, it does not appear that corrosion or other factors have affected the mechanical strength characteristics of the 48-inch diameter cast iron pipe.

Internal camera inspection of 12 percent (approximately 3,000 feet) of the 72-inch steel pipeline alignment did not reveal evidence of any structural deficiencies. The cement mortar lining was intact, but in some locations found to be leached to some degree.

Metallurgy

For the 48-inch cast iron pipe, tensile test results were consistent with cast iron pipe from 1922 in accordance with AWWA 7C.1. No cracks were observed and microstructure features did not suggest a degradation of the pipe material. No graphitization of the pipe wall was found and the chemical composition was found to be typical for grey cast iron pipe.

For the steel pipe, visual testing, penetrant testing, and magnetic particle testing of the factory spiral welds indicated that they had acceptable profiles and reinforcement in accordance with current welding standards (AWS D1.1-2006, Section 5.24.4). However, radiographic testing revealed that portions of these welds were unacceptable due to their porosity, incomplete fusion and penetration, and inclusion of slag. Although these defects are notable, it is not believed they are related to past incidents of full-circumferential brittle failure of the pipeline.

Charpy V-Notch impact test data strongly suggested that the steel used in the 72-inch pipeline lacks adequate fracture toughness to prevent brittle failure within its normal range of operating temperatures. Since all failures have occurred during winter months when water temperature is lowest, the loading mechanism at failure is likely thermal-induced longitudinal tension or perhaps a combination of thermal-induced static tension and a dynamic event. The extremely low fracture toughness of all portions of the pipeline below 40 degrees F makes suggests that joint rehabilitation on this brittle metal would likely not be an effective solution and could result in additional failures resulting from the poor metallurgical qualities of the pipe barrel.

Rehabilitation Options Considered

The following options were considered:

- Cement Mortar Lining--non-structural renovation method for water piping.
- Epoxy Lining--does not improve the structural integrity of the host pipe.
- Slip-Lining--flexible thermoplastic liner directly inserted into a host pipe and well suited to rehabilitating existing pipelines with poor structural integrity.
- Modified Slip-Lining--structural renovation technique whereby the inserted thermoplastic pipe is temporarily deformed to allow clearance; after placement, tube is expanded in place and provides close-fit liner to the host pipe annulus.
- Cured-in-Place Pipe Lining--structural renovations that involve the insertion of a resin-impregnated tube in the host pipe.
- Internal Joint Seals--renovation technique designed to seal the inside surfaces of leaking joints.
- Pipe Bursting--trenchless replacement technique whereby the replacement pipe is commonly high-density polyethylene (HDPE).

- Carrier Pipe in Casing—straight runs of failing host pipe with sufficient structural capacity can serve as casing pipes for new carrier pipe.
- Same Trench Replacement--requires that the existing pipe in an easement or right-of-way (ROW) be excavated, demolished, and replaced with a new pipe.
- New Trench Replacement--a new pipe installed parallel to an existing line.
- Insitu Structural Repair--an internal butt-strap method to bypass the existing bell and spigot lap joints and relieve axial thermal tension.

Ranking of Rehabilitation Alternatives

In order to produce a set of viable solutions for rehabilitation of the City of Atlanta raw water transmission system, the engineering team developed a step-wise method for comparing alternatives according to four evaluation categories:

- Selection of Rehabilitation Technology
- Hydraulic Performance
- Construction Cost
- Construction Impacts

These 4 categories were weighted equally at 25 points. Each category was then scored 0 – 25 for the various rehabilitation techniques according to their applicability and effectiveness. The highest total score for a particular rehabilitation alternative would be 100. The total scores are shown in Tables 3 and 4.

Cast Iron Pipes

The results of the condition assessment indicated that the cast iron pipelines are at low risk of structural failure. The major risk of failure for the cast iron pipelines is leakage from the joints and subsequent bedding failure. Application of the selection method indicated that non-structural relining alone was not sufficient to provide the most effective long-term solution. When combined with internal joint seals, such a combined rehabilitation method for these pipes is the most feasible. Structural liners were also feasible, but brought an associated reduction in capacity. Pipe bursting was considered possible, although it is used rarely in larger pipe sizes. Finally, parallel trench replacement with a single new pipeline of equivalent capacity was considered.

The final ranking of the alternatives is shown in Table 3:

Joint Seals/Lining	100.0
HDPE Slip-Lining	83.5
Close Fit HDPE Lining	81.4
HDPE Pipe Bursting	70.1
CIPP Lining	69.9
New Pipe - Parallel Trench	69.8

Table 3. Summary of rehabilitation method ranking (cast iron mains).

72-Inch Steel Pipeline

The results of the condition assessment confirmed that the 72-inch steel pipeline was built with steel of inferior quality. There is a continuous risk of failure due to poor metallurgical qualities of the pipeline. Use of the selection method revealed that only a limited number of rehabilitation practices are applicable to 72-inch diameter steel pressure pipe. The basic requirement for a structural rehabilitation narrowed the possible methods to in-situ structural repair, re-lining with a structurally independent carrier pipe, and parallel trench replacement.

It was concluded that in-situ structural repair (butt strap repair of the joints) should not be attempted due to the unacceptable risk of additional failures resulting from the poor metallurgical qualities of the pipe barrel. This left only two options.

Standard casing practice for 72-inch lines is to install a 60-inch carrier, and hydraulic modeling of the baseline rehabilitated system indicated that the replacement line would still provide acceptable hydraulic capacity, in combination with rehabilitated cast iron lines. Therefore, a 60-inch line was evaluated for both the carrier pipe and parallel trench options. It should be noted that a parallel trench pipe of greater diameter could be installed, adding additional capacity. However, in order to have a common basis for cost comparison, a 60-inch pipeline was assumed.

The final ranking of the two alternatives is shown listed in Table 4:

Carrier Pipe in Ex. Casing	89.1
New Pipe – Parallel Trench	76.3

Table 4. Summary of rehabilitation method ranking (72-inch steel main).

Hydraulic Analysis

A steady-state hydraulic model was used to verify the capacity of the post-rehabilitation pipelines in the evaluation of alternatives. While some linings would not create a significant reduction in cross-sectional area, replacement piping using the existing mains as a carrier would reduce pumping capacity. The models were also used in developing design pressure for the various pipelines, and in optimizing pressure rating requirements for different sections of the same pipeline.

In this project, the hydraulic model was used to do the following:

- Determine design pressures for use in evaluating rehabilitation alternatives.
- Check alternatives against firm pumping capacity.
- Provide accurate take-offs of pipe segment lengths for cost estimating.
- Export the database to SURGE transient modeling software.

H2OMAP software by MWHSoft was used to construct the model.

In order to establish a baseline for the capacity of a rehabilitated system, a steady state model was developed assuming a reasonable ”worst case” scenario. Slip-lining the existing cast iron pipes with HDPE piping rated at [spell out SDR?] SDR 11 (pressure rating 160 pounds per square inch [psi]) would produce the greatest reduction in cross-sectional area in those lines. The worst case scenario for rehabilitation of the 72-inch steel line was modeled as lining the existing host pipe with a 60-inch carrier pipe.

Conclusion--Recommended Rehabilitation Methods

72-inch Steel Pipeline

The results of the condition assessment tasks indicate that the existing 72-inch pipeline is at risk of ongoing failures resulting from the use of inferior brittle steel during fabrication. Corrective action in the form of in-situ structural rehabilitation is feasible, but has the risk of continued failures in the remaining (original) pipe material. Therefore, a structural repair to the existing pipe is not recommended.

The in-situ rehabilitation options for the 72-inch steel pipeline are limited compared to the options available for smaller-diameter pipe. None of the newer materials used in pipeline rehabilitation (such as HDPE slip-lining and cast-in-place liners) have practical application or proven records in water service in this line size. However, the 72-inch steel pipeline does have sufficient internal area to provide a viable conduit for the installation of a new, fully independent pipe. While such a project would not be without challenges, the large internal diameter provides ample working space. The linear nature of much of the alignment also makes the option attractive in terms of constructability.

Treating the existing pipe as a standard casing, the preferred new pipe size would be 60 inches. This selection of pipe size allows for price competition from the three common materials used in the fabrication of large diameter pipe: ductile iron, carbon steel, and pre-stressed concrete cylinder pipe. The annulus between the new carrier pipe and the existing 72-inch steel pipe is often filled with lightweight grout, the cost of which can be substantial in the quantities required for this project. It was recommended that casing spacers be considered as a lower-cost alternative.

Cast Iron Pipelines

The results of the condition assessment indicate that the 30-inch, 36-inch, and 48-inch pipelines are in good structural condition. The pipelines have provided reliable service with a limited history of service outage due to breaks. Heavy corrosion or excessive graphitization of the pipe barrel were not noted during the current study, an indication that no advanced material deterioration is occurring in the areas observed. Based on the lack of evidence that the cast iron lines are failing structurally, it is recommended that they remain in service but undergo an extensive rehabilitation to maximize their service life.

Based on estimated costs, and construction impact considerations, the primary recommended rehabilitation alternative in this case is the installation of internal joint seals. The pipeline should then be relined with a National Science Foundation (NSF) approved spray-applied epoxy lining. Application of a new cement mortar lining was rejected due to a service life that is limited to 50 years.

It was recommended that construction documents allow an alternate bid for slip-lining. There are materials and techniques that could be applied to slip-lining cast iron pipes in the sizes under consideration. This alternate bid would allow a larger number of vendors to bid on the job, and the cost of the project is sufficient to encourage innovative approaches for the composition, fabrication, and installation of liners.

The Perspective of the Small Diameter Subterrene Rock-Melting Drills Used for Trenchless Pipeline Installation

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Abstract

Trenchless methods are frequently utilized to install pipelines in some difficult conditions, such as loose sand, non-cohesive soil and fractured formations, which would cause hole-wall collapse, over-pull back force and/or even break the drill rods. It would be very troublesome and costly to use Horizontal Directional Drilling methods to construct underground pipelines in these kinds of strata.

During the last several years the authors demonstrated practical applications of drilling using an electrically-heated graphite, tungsten, or molybdenum penetrator (subterrene drill) that melts a hole as it is slowly pushed through the rock or soil. The molten material consolidates into a rugged glass lining that prevents borehole collapse; minimizes the potential for cross-flow, lost circulation, or the release of hazardous materials without casing operations; and produces no cuttings in porous or low density formations. Because there are no drilling fluids required, the rock melting approach reduces waste handling, treatment and disposal. Because the penetrator does not need to rotate, steering by several simple approaches is considered quite feasible. Melting is ideal for making a hole in alluvium and other poorly consolidated soils since the formed-in-place glass liner stabilizes the hole. Because of the relatively low thermal conductivity of rock and soil materials, the heat-affected zone beyond the melt layer is very small, <1 inch thick.

This paper will present the in-door experiments of the rock melting approach to make a hole and discuss the perspective of this hole-making method to be utilized in underground pipeline trenchless construction.

Key Words

Subterrene drills, Trenchless Technology, Pipeline Installation, Drilling, Rock-melting

Introduction

Trenchless Technology is the science of installing, repairing or renewing underground pipes, ducts and cables using techniques which minimize or eliminate the need for excavation (ISTT, 1998). Trenchless methods are usually utilized to install pipelines in some unfavorable conditions, such as loose sand, non-cohesive soil and fractured formations, which would cause hole-wall collapse, over-pull back force and/or even break the drill rods. It would be very troublesome and costly to use Horizontal Directional Drilling methods to construct underground pipelines in these kinds of strata. During the early and mid-1970's the Los Alamos National Laboratory demonstrated practical applications of drilling and coring using an electrically-heated graphite, tungsten, or molybdenum penetrator that melts a hole as it is slowly pushed through the rock or soil. The advantages of the new drilling technology can be summarized as follows (Cort, et al，1994):

- The molten material consolidates into a rugged glass lining that prevents hole collapse;
- Minimizes the potential for cross-flow, lost circulation, or the release of hazardous materials without casing operations;
- Produces no cuttings in porous or low density formations.
- No drilling fluids are required; the rock melting approach reduces waste handling, treatment and disposal.
- Can be used in caliches, clay, alluvium, cobbles, sand, basalt, granite, and other materials. Penetrating large cobbles without debris removal was achieved by thermal stress fracturing and lateral extrusion of portions of the rock melt into the resulting cracks.
- Can be used to drill both horizontal and vertical holes in a variety of diameters.

Rock melting drilling can be applied to many down-hole operations:

Geothermal industry. Rock melting is well suited for use in the high temperature environment associated with geothermal sites. The glass liner can serve to reduce or eliminate down-hole cementing and re-drilling operations.

Hydrocarbon industry. The ability to form an impermeable sheath to line the borehole can greatly reduce the problems of borehole sealing and collapse.

Environmental remediation. Rock melting drilling offers several advantages: (1) no fluid lubricants are involved that might cause cross-contamination; (2) the in-situ glass lining provides immediate borehole sealing and stabilization; (3) the process is most useful in porous consolidated or unconsolidated rocks, where the use of conventional methods is difficult; (4) the resulting glass sheath has a low thermal conductivity so that temperature gradients in the vicinity of the borehole are high and the surrounding rock substrate is affected by the rock melting within only about ten centimeters from the borehole.

Horizontal directional drilling (HDD). HDD is being used increasingly for underground pipeline installation. Most of these drilling environments involve porous and loosely consolidated materials in which rock melting drilling can produce linings in-situ during drilling to immediately stabilize the boreholes.

Because of these great benefits for making a hole in some difficult conditions, the authors started their research work on rock-melting drilling technology and its application in 2001. After a couple of year's intensive research, they have improved the small diameter rock-melting drill tools, got to know the basic working principle, and have exploited its potential application field. It can be a good trenchless method to install underground pipelines under some very complex, or difficult soil conditions.

The Working Principle of Rock Melting Drilling Method

Rock melting is a promising drilling technology for stabilizing boreholes in unstable rock formations or in unconsolidated materials. The drilling system conventionally uses a penetrating bit of, e.g., molybdenum, electrically heated to 1,600° C. to melt a hole, typically two to three inches in diameter, as the penetrator advances. In porous minerals, the molten material is displaced around the sides of the penetrator, forming a glass-like lining that prevents borehole collapse. The mechanical integrity of the glass lining that is formed determines its ability to form a rigid impermeable glass casing in the borehole. Current glass forming afterbodies solidify the melt phase into a glass lining by quickly cooling the liquid from the penetrator tip temperature of about 1600° C. to the gas cooled temperature of 200° C. in approximately 10 minutes. This rapid quenching of the molten rock glass freezes thermally induced strains in the glass, which results in cracking of the glass lining (US Patent 700954).

According to Tang (2000), when drilling in loose rock strata, the thickness of the glass-like lining ΔR depends on the radius of the borehole and related to the property of the soil/rock to be drilled. The relationship can be expressed in the equation as follows:

$$\Delta R = r_0 \bullet \sqrt{\frac{\rho_{granite} - \rho_{rock}}{\rho_{granite} + \rho_{rock} - 2\rho_{rock}\left(1 - G - W_{rock} - W'_{rock} - E\right)}}$$

Where: r_0 — the radius of borehole (m), $\rho_{granite}$ — the density of granite (kg/m^3), ρ_{rock}—the density of rock (kg/m^3), W_{rock}—the percentage of the water in the rock under the natural conditions (%), W'_{rock} —the moisture of the rock (%),—the percentage of volatile matter in the rock under high temperature conditions (%), G—the percentage of the organic and other flammable matter in the rock (%).

Experimentation

The authors have done some research works on rock melting drilling technology in the laboratory. The test drilling device (shown in Figure 1) includes mainly a rock melting penetrator (its section view is illustrated in Figure 2), a drilling rig, a DC adapter (~70V, ~200A), etc. The layout the whole system is shown in Figure 1.

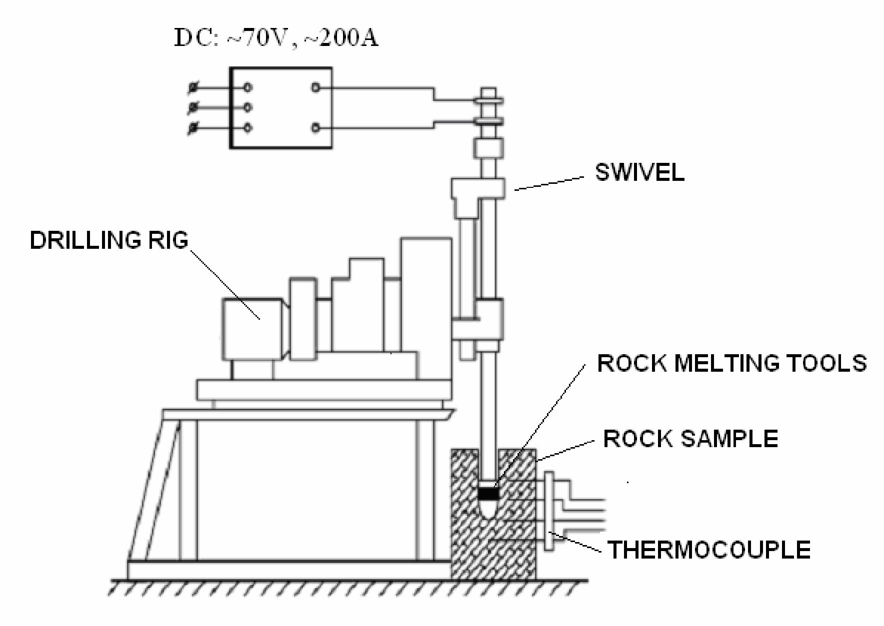

Figure 1. The rock melting drilling test system

The rock melting penetrator shown in Figure 2 is the key component of rock melting drilling technology. But the core elements for the rock melting penetrator are the thermal decomposition graphite and the outer shell of the penetrator. When the electrical current passes through the central rod (anode), electrode connection, thermal decomposition graphite sheets, the protection shell (outer shell) of the penetrator, and the drill rod to make a circuit, the thermal decomposition graphite can generate heat, which can be transmitted onto the outer shell of the penetrator and the surrounding soil through radiation, conduction, and convection, etc. When the surrounding soil/rock obtain enough heat or the temperature is 1100-1600°C (depending on the type of the rocks or soils), the rock/soil begins to reform and melt.

After cooling, the melted rocks or soils can be converted into glass-like matter and serves as the casing/liner of the borehole. But the liner is clearly layered radically. The thickness of the liner can be 25~30mm (1-1.2 inch), and divided into three different zones: lower temperature zone (lower than 1000° C), medium temperature zone (between 1000 to 1500° C), and higher temperature zone (shown in Figure 3). In the higher temperature zone, the original structure of the rock is almost damaged entirely, and formed into brittle glass-like material with higher hardness. In the medium temperature zone, the rock is form into ceramic-like material with higher strength. While in the lower temperature zone, under the heating action the rock changed its properties, with higher chemical sensitivity, but lower mechanical properties.

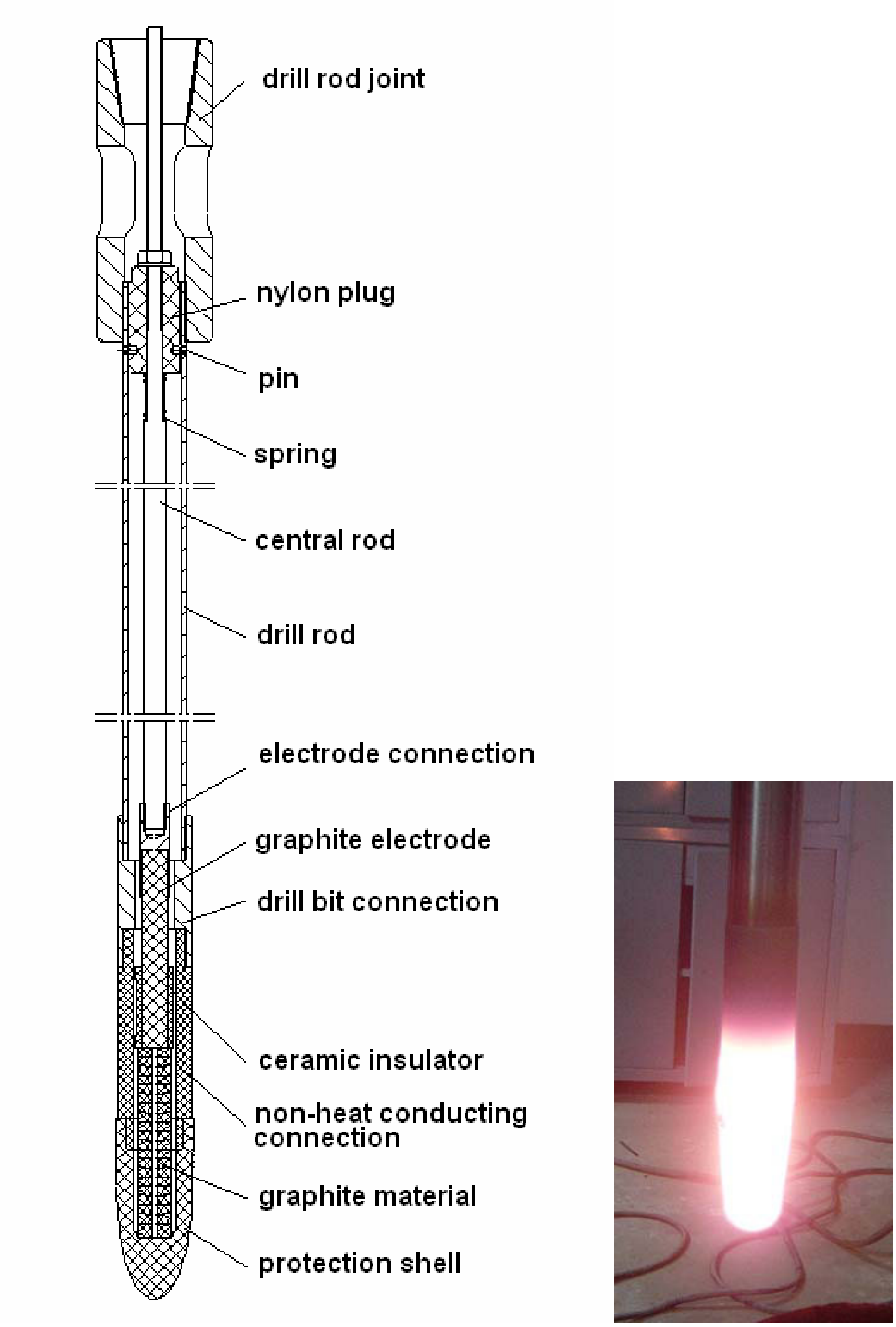

Figure 2. The rock melting penetrator

The temperature field around the penetrator

During the rock melting penetrating, the temperature around the penetrator is one of the most important factors affecting the penetration rate. So the authors have done some research work to find the rule of distribution of the temperature field around the penetrator during the experimentation. The contacting thermocouples are used to measure the temperatures around the penetrator. The layout of the temperature measuring system is shown in Figure 4. There are 3 measuring points set up around the penetrator radically and axially respectively, so the measuring system provides a total of 10 measuring points around the penetrator. According to the real time temperatures measured by each point, the temperature distribution curve around the penetrator can be obtained (as shown in Figure 5).

Figure 3 The glass-like casing/liner of the borehole

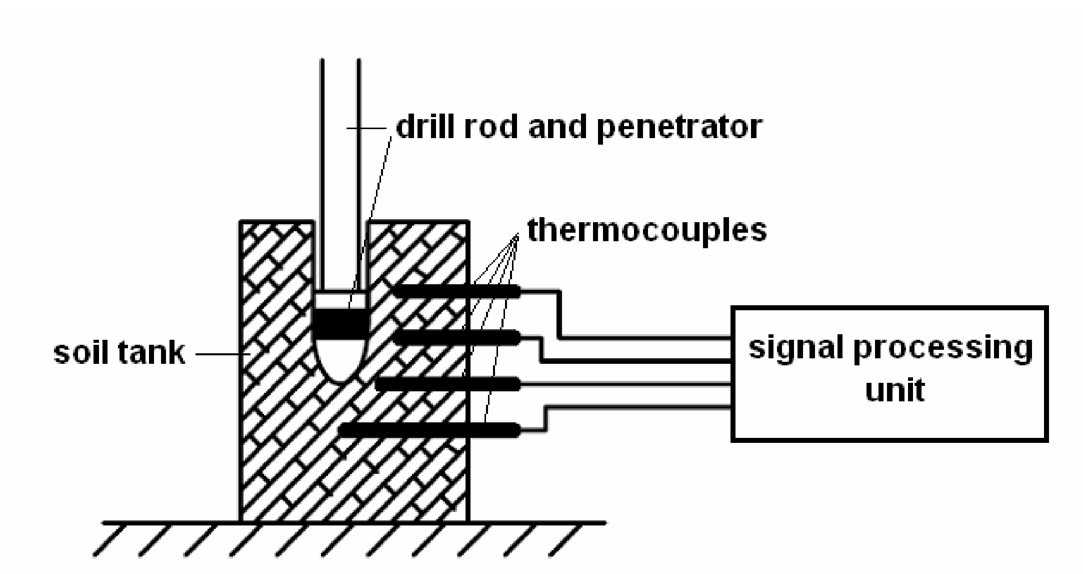

Figure 4. The layout of the temperature measuring system

Experimental procedure. 1. Set up the working electric current through the penetrator I=140A, when the temperature on the surface of the penetrator t reaches 1318, thrust the penetrator into the soil/rock to make hole. 2. During the drilling process, in order to get a real temperature distribution field and a good quality glass-like casing/liner, a lower penetration rate should be kept. Based on the experimental data on fine sand formation conditions, a temperature-distance equation can be obtained as follows:

$$t = 1182.4 \cdot e^{-0.1431x}$$

Where: t —temperature of the measuring point, ℃

x —the distance from the measuring point to the penetrator surface, mm.

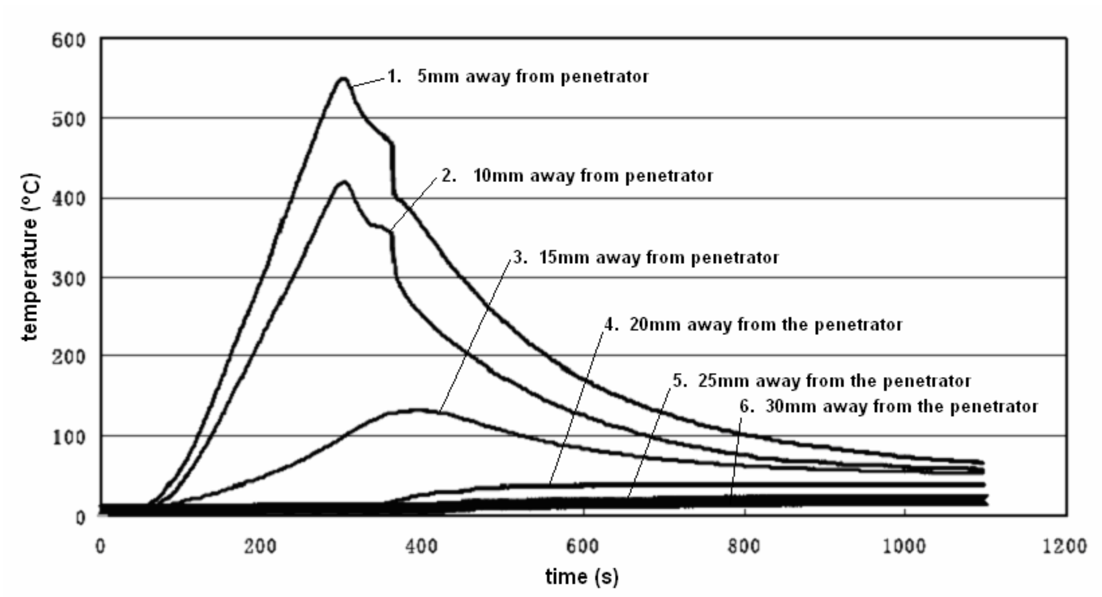

Figure 5. the temperature distribution curve around the penetrator

The perspective of rock-melting drill to be used for Trenchless pipeline installation

The rock melting drill offers great benefits for making holes in some unfavorable soil conditions, such as loose sand, non-cohesive soil and fractured formations, which would cause hole-wall collapse, over-pull back force and/or even break the drill rods. If the rock melting drilling technology can be used to install pipeline in these conditions, an in-situ casing/liner can be produced immediately after the penetrator passed and the borehole wall can be stabilized and enhanced. So the pipeline can be easily installed through the horizontal borehole. The schematic diagram of using rock-melting to make horizontal hole is shown in Figure 6.

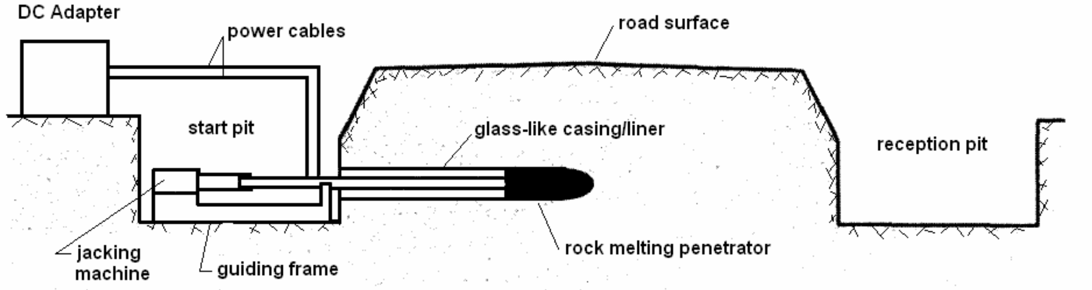

Figure 6. The schematic diagram of using rock-melting to make horizontal hole

Conclusion and discussion

Rock-melting is a non-conventional drilling method, which has many unique benefits, such as no complicated equipment is needed, can form in-situ glass-like casing/liner of the borehole, so no casing is needed to enhance the borehole wall to avoid its collapse when using this method to construct underground pipeline in some unfavorable soil conditions.

The authors have already developed a rock-melting penetrating system which works perfectly in sand or soft formations. Through intensive research, they have understood the working principle, temperature distribution field around the penetrator, etc.

But all the research work on using rock-melting method to install underground pipelines without trench is just at the beginning, there is still a lot of work remaining to turn this research into practical application.

References:

ISTT. (1998). “Trenchless Technology Guidelines”. London

Cort, George E., Goff, S.J., Rowley, John C., et al.(1994). “Rock melting approach to drilling”, Proceedings of the Energy-Sources Technology Conference, New Orleans, LA, USA

US Patent. (1998). No. 700954, “Rock melting tool with annealer section”. Issued on April 7, 1998 filed on 1996-08-21

Tang, F.L. (2000). Rock melting drilling technology and its application in constructions. Geological Science and Technology Information. (in Chinese)

Practical Engineering Considerations for developing a free-swimming tool for RFEC/TC inspection of PCCP transmission mains in a live operating environment

Xiangjie Kong[1], and Brian Mergelas[2]

Abstract

Condition assessment of Prestressed Concrete Pressure Pipe (PCCP) has evolved over the past 10 years mainly due to the development of the Remote Field Eddy Current / Transformer Coupling (RFEC/TC) inspection technique. By conducting RFEC/TC inspections to determine the actual condition of their pipelines PCCP owners and operators can make informed decisions regarding strategic repairs and selective rehabilitation. However, RFEC/TC inspections can, in some circumstances, be a challenge to conduct.

Various designs of manned tools and tethered systems have been successfully used in the field but they require dewatering and/or depressurizing which can be costly and prevent the inspection of pipelines that can not be taken out of the service. To overcome these challenges a free-swimming tool for RFEC/TC inspection of PCCP in a live operating environment is currently being developed. This paper describes practical engineering considerations for such a development.

Background

Prestressed Concrete Cylinder Pipe (PCCP) has been widely used for the large-scale transmission of water and wastewater for more than half a century. PCCP is a skillfully engineered composite that exploits concrete in compression and steel in tension to create an economical, efficient and long lasting product. However, PCCP occasionally ruptures due to corrosion and breakage of the high-strength prestressing wires that reinforce the pipe. The initial rupture, and the large amounts of high-pressure water released, can lead to hazardous local conditions, public safety considerations, significant property damage and extensive service interruption. In addition to costs for repair or replacement, water supply system operators may be faced with revenue loss, water rationing and legal liability.

Pipeline operators who wish to avoid these consequences and ensure the continued safe and economical operation of their PCCP pipeline are actively seeking solutions that provide an accurate evaluation of their aging

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infrastructure. In response to this need the Remote Field Eddy Current / Transformer Coupling (RFEC/TC) technique was developed by Prof. David L. Atherton at Queen's University. The system was patented based on his work in the late 80’s and 90’s and offers water supply system managers the best possible condition assessment information by providing the location, distribution and number of wire breaks anywhere along the length of their PCCP.

To date the RFEC/TC technique has been used to successfully inspect over 4,000 km of large diameter (36" diameter and greater) PCCP pipelines. The inspection results give pipeline operators the information they need to repair, replace or monitor individual pipes. As a result they can maximize the safe and economic lifetime of their pipeline. Some operators have reported savings of tens of millions of dollars by acting on the results provided by a RFEC/TC inspection.

Currently most RFEC/TC inspections are conducted by walking or riding a self-contained, manually operated inspection unit through a pipeline. There are several successive generations of walking and riding styles of RFEC/TC inspection tools which are capable of inspecting pipes ranging from 36” to 252” in diameter. All of the required equipment can be disassembled, passed through the 18” or larger access manhole, and then reassembled inside the pipe. Using manned tools pipelines under investigation need to be dewatered and the pipes have to be larger than 36” so that safe access for the inspection crew is ensured.

Building on the success of manned inspections the first unmanned RFEC/TC system was developed in 2005. This system is a four-wheel drive vehicle with on board RFEC/TC electronics which is remotely controlled with a cable. It also has CCTV capability so that internal pipe conditions can be observed and recorded. It is able to inspect PCCP lines as small as 24”. Full dewatering is not necessary but depressurizing the entire line and dewatering the access areas is required. The system can be inserted and retrieved through a 20" or larger manhole although, depending on the exact access layout, an open cut may be required for pipes smaller than 30” in diameter.

During the inspection both visual and RFEC/TC signals are transferred through the tether cable in real time so that the tool can be navigated and data quality can be monitored. The cable can be up to 4000’ long, which means that the inspection distance can be up to 8000’ (combining upstream and downstream runs) with one insertion. Since its commercial introduction in 2005 a variety of small diameter PCCP lines have been successfully inspected using this system. Recently, a smaller version has been developed capable of inspecting lines as small as 16” in diameter. This extends the range of RFEC/TC inspection to all PCCP being manufactured.

All of the previously mentioned inspection systems require the PCCP line to be taken out of service which is very costly, inconvenient and sometimes simply not possible for pipeline operators. Ultimately, a free-swimming tool is needed for condition assessment of PCCP lines in a live operating environment. The system should be capable of being inserting into a live pipeline, traveling in pressurized water flow while conducting RFEC/TC inspection and then being retrieved from the line, all without taking the line out of the service. There are many engineering challenges for such a development and some practical considerations will have to be evaluated.

Access

A RFEC/TC inspection starts with the tool entering the pipe and ends with the tool exiting the pipe making the access one of the most important factors of the tool's design. Unlike some oil and gas pipelines where full bore launchers and receivers are often built enabling smart pigs to be inserted and retrieved, water pipelines have very limited accesses specially designed for inspection purposes. In most cases the access to the PCCP line is through manholes which can have a variety of designs and constructions. Manholes range in size from 16” to 36” in diameter and while most manholes are round they can also be oval in shape. Manholes are typically oriented in the vertical direction but horizontal manholes are also commonly encountered. These non-standardized accesses pose a great challenge for inspection of live water pipelines. To date, manual insertion and retrieval are used in all RFEC/TC inspections, including the unmanned operations where the vehicles are manually inserted and setup in the access area. To conduct a RFEC/TC inspection in a live condition a pressurized launching and receiving mechanisms will have to be developed.

The Sahara leak detection system can be inserted and retrieved through a 2” tap in live conditions. However, due to the current size and special requirements of RFEC/TC inspection system, 2” is too small.

Having full pipe diameter launchers and receivers, as with the piggable oil and gas pipelines, is the most desirable option from the RFEC/TC tool design point of view but is an expensive option for the pipeline operator. For a very long straight line (hundreds of miles) without internal bore restrictions (i.e. butterfly valves), it may make economic sense, however, this is rarely the case in water transmission mains and full bore access is not practical.

The most cost-effective solution is to design and build a standardized pipeline access which could be much smaller than actual pipe diameter. This standard access should be not only the key design requirement for the free-swimming tool, but also the guide for utilities to ‘upgrade’ their pipelines for inspection. It is recommended to include the standard access in new installations for future inspections.

Pipeline features

Once in the pipe the tool has to travel, while collecting RFEC/TC data, to the receiving point. Many pipeline features, such as reducers, large outlets, elbows, valves, etc..., may pose challenges for the free swimming tool to pass. The oil and gas pipeline industry has made specific allowances for the passage of smart pigs, such as installation of new valve stations, tee bars, bypass fixtures etc..., which are not present in most water transmission pipelines. While it is expected that the water industry will modify transmission mains as inspection tools evolve, careful design considerations will have to be made to ensure that the free swimming tool can pass through as many existing pipeline features as possible.

As the tool will be inserted through an access smaller than pipe diameter it is expected that the tool be flexible and should be able to pass reducers and accommodate small variations in pipeline diameter and roundness. Engineering design and modeling will be conducted to ensure that the tool can pass through the most commonly found bends and elbows. For large outlets, such as T's and Y's, flow should be shut off to prevent the tool from going off the main line. Butterfly valves, often used in transmission mains, are likely not passable and standard accesses should be installed on each side of the valve.

RFEC/TC requirements

RFEC/TC has been proven to be the most effective condition assessment technique for PCCP. It is accurate in detecting and quantifying wire breaks but sensitive to sensor configuration and movement. The free-swimming tool has to maintain the effectiveness of the RFEC/TC inspection by meeting its essential requirements.

In a RFEC/TC system the electromagnetic field that is sent from the transmitter to the receiver is amplified by the prestressing wires. If there are broken wires the received signal will be disturbed to some degree. By quantifying the amount of disturbance to the signal the number of wire breaks can be predicted.

From years of experience we know that RFEC/TC has the best performance when the transmitter and receiver are placed with no axial separation within the pipe and a large radial separation. This means that optimal sensor structure will be close to a full pipe diameter in length. Since the tool needs to go through a standard access, which generally will be much smaller than the actual pipe diameter, it will be challenging to design a sensor structure that expands to the full pipe diameter after insertion, maintains its configuration in a pressurized water flow during the inspection and then contracts the structure for retrieval. Sophisticated control systems and the correct size of access will be needed to achieve these under live conditions in order to collect best possible data.

Other considerations

Communication: It is desirable to have real time communication so that the data can be monitored and tool can be tracked at all times. A tether with fiber optics cable can achieve that, however, it may complicate the launch and retrieval design and limit the inspection distance. For a truly free-swimming tool, without a tether, it is possible to have a ping station every few hundred feet above ground along the line during the inspection so that the tool can still be tracked intermittently.

Prescreening: As we have learned from field experience there are sometimes unknown features encountered inside the pipeline like valves not shown in any drawing or cross bars left over from construction. These may prevent the free swimming tool from passing or even damage the tool. To avoid these situations a prescreening tool is considered. Sahara, a tethered system capable of being inserted and retrieved from a 2" tap, has been used in water transmission mains for leak detection in live conditions. Recent development has shown that it is possible to add CCTV capacity which could be the best solution to prescreen the line before inserting a free swimming tool.

Water contamination: Because the free-swimming tool will be used in live pipelines the system should have minimal and acceptable effects on water quality. All industry standards and regulation must be followed. The system should be totally watertight and reliable so that not only it functions well under water but, more importantly, no part will be left in the pipe after inspection. During the inspection the contact between tool and pipe wall should be minimized to reduce the possibility of stirring films and debris often seen in water transmission mains. Depending on the utility, public notices and/or flushing afterward may be required. This is not uncommon for in-line inspection and should be manageable from the utility's perspective.

Conclusion

RFEC/TC has been proven to be an effective condition assessment technology for PCCP. While the technique works for almost all types of PCCP it is a huge challenge to actually conduct RFEC/TC inspections for many PCCP lines. Various designs of manned tools and tethered systems have been successfully used in the field but they require dewatering and/or depressurizing which can be costly and prevent the inspection of pipelines that can not be taken out of the service. Ultimately, the goal is to develop a free-swimming tool capable of conducting a RFEC/TC inspection in the live condition so that service interruption can be minimized, the total inspection cost can be reduced, and allow virtually all PCCP lines to be inspected. Engineering considerations for the development of such a tool are evaluated.

Regarding in-line inspection, the water industry is several decades behind the oil and gas industry where inspection is regulated, smart pigs are available, and many lines were, and virtually all new lines are, designed with inspection in mind. To develop a practical free-swimming tool it is important to work with water utilities to standardize the access and inspection protocol for in-line inspections. Efforts and expenditures should be optimized between existing system upgrades (for inspections) and tool advancements. For new installations it is recommended to add standard access and where possible ensure clear passage through the line to facilitate future inspections.

References

B. Mergelas and D.L. Atherton, “In-line Electromagnetic Inspection of PCCP”, Proc. Pipelines in the Constructed Environment, San Diego, 23-27 August, 1998.

B. Mergelas, X. Kong and D.L. Atherton, “NDE Inspection of PCPP using RFEC/TC’, Proc. Pipelines 2001 Int. Conf., San Diego, 15-18 July, 2001.

B. Mergelas and X. Kong, “Best Practices in the Condition Assessment of Water Transmission Pipelines”, Underground Infrastructure Advanced Technology Conference, Washington, D.C, December 6-7, 2004.

Development of Keyhole Pipe Tapping and Plugging Tools

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Abstract

This paper is an overview of a joint product development program conducted by TDW and GTI in 2005. GTI originally contacted TDW in 2004 and indicated there was a need for hot tapping and plugging via keyholes. The term Keyhole, as used in the gas piping industry refers to small round or square access ports whereby buried pipelines can be accessed.

The main goal of the project was to develop and demonstrate Hot tapping and plugging (stopping of the flow of the product in the line) could be done on buried pipes up to 6 feet below the ground via 18" cored holes in roadways, sidewalks, etc. This need was driven by increasing excavation and repaving costs, traffic problems, and safety issues.

As stated, this technology would result in less job costs to the utility company by reducing the costs associated with pipeline excavation and street repairs. A safer system would also exist since personnel would not be subject to cave-ins.

Introduction

In Keyhole Technology a circular cutout, ranging from 6" to 24" in diameter is cored out of the street and the "core" of pavement is removed and set aside. The pipeline is subsequently exposed by removing the dirt from above it. Once the pipe is exposed, it is cleaned using a scraping or sandblasting device. Long handled tools can then be used to attach a bolt on fitting. Fittings typically consist of a hinged body or a separate upper and lower half, with o-ring or sheet gasket to prevent leakage once the pipe is tapped into through the fitting's outlet (Figure 1).

Figure 1

Special long handled tools are then required to attach the fitting, and secure it to the pipe. (Figure 2).

Figure 2

Once installed and pressure tested (Figure 3), an isolation valve is attached (Figures 4 and 5), so the pipeline may be punctured or "tapped" through the fitting without the loss of pipeline product.

See following page for Figures 3, 4 and 5

Figure 3

Figure 4

Figure 5

Tapping into the pipe is performed by a drilling or tapping machine connected to the valve (Figure 6).

Figure 6

The machine has an extendable and rotate-able bar that advances and rotates a hole-saw or pilot drill. The connection to the valve is typically a flange joint or quick connect adapter.

To stop the flow of a broken pipeline, or bypass around a damaged section, make repairs to a valve, etc, a flow stopping device is inserted upstream and or downstream using a machine with an extendable bar. The machines are known as Stopple setting machines or Jackscrews (Figure 7 thru 9).

Figure 7

Figure 8

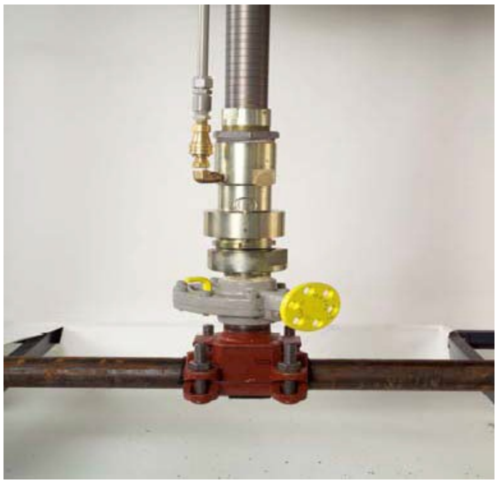

Figure 9

Once the work has been done on the pipeline, the completion plug will be installed into the fitting neck so the valve can be removed (Figure 10).

Figure 10

Once the completion plug is installed, pressure can be blown down above the valve and valve removed. The final step is to install the completion cap on the fitting (Figure 11).

Figure 11

It should be noted that the machines that tap and plug the line must be light enough and long enough to be lowered down by one man, and easily and safely attached to, and detached from the isolation valve. Lightweight tools are also required that are sufficiently long enough to reach a pipe buried 6 feet below grade.

Design Approach

The actual design of equipment began after several trips by the development team to various keyhole demonstrations and workshops around the country. Once the preliminary research was completed and the keyhole market was fully understood, the design team was formed.

The team quickly decided the best approach to get to market the fastest, was to modify some existing products (machines and bolt on fittings) and simply develop new tools which would allow these machines and fittings to be installed via a Keyhole.

Several existing tapping machines were evaluated. Because the scope of the project was 2" steel mains, an existing lightweight machine with 16" tapping stroke was chosen. Besides being able to reach through the valve and make the hot tap, it could also be used to install a completion plug in the neck of the fitting just below the valve. Completion plugs allow the isolation valve to be removed.

To allow this shorter machine to be used below the surface by a man standing on the surface, a simple extender apparatus was designed.

The plugging machine which is used to insert a stopper into the flow of the pipeline, was an aluminum version of an existing steel machine. Besides making it from aluminum, it was longer than the steel machine so it would reach above ground also. To solve the problem of venting gas (prior to removing the machines) a long piece of tubing, known as “blow-down” piping was routed to the surface on both machines.

The Keyhole fitting was split in half, so it could be easily clamped onto the pipe using a special fitting installation tool (Figure 1). The top seal style fitting utilizes a buna gasket which straddles the pipeline similar to a saddle.

Several long handled tools were created to attach the fitting, attach the valve, make down-hole measurements, plug the pipeline, blow down sections of isolated line, tighten fitting nuts, torque fitting nuts, etc.

Quick-connect adapters were developed to allow the machines to be easily attached and detached from the isolation valve.

The maximum working pressure of the system is 100 PSI, mainly limited by the aluminum body of the jackscrew machine. The size of pipe that can be worked on at this time is 2” API or 2-3/8” OD only.

The weight of the tools ranges from 1 lb to 45 lbs. The fittings weigh 9 lbs and 12 lbs for the straight neck fitting and the 3 way tee respectively. This system was designed for use on natural gas distribution lines only.

Conclusion

The 2” HT&P system that was developed during this particular project is for 2” std weight steel pipe only, but will work on any pipe 2-3/8” diameter pipe, with schedule 20 or greater wall thickness. This includes, but is not limited to, pipe materials such as Polyethylene, PVC, Cast Iron, Stainless Steel, etc.

In Summary, there are many types of work being done today via Keyhole access such as; installing leak repair clamps, adding new service lines, service cut offs, cathodic protection, bell joint encapsulations, launching of pigs and inspection devices, etc.

The system of equipment and tools developed during this program, met all requirements by GTI and their sponsors and performed to the specifications established early in the program by TDW. Using these tools to work on 2” steel pipes via Keyholes, will reduce excavation costs, crew sizes, and personal injuries. More importantly, it will also result in a safer, more reliable transmission infrastructure.

Table 1, below, is a summary of the main products developed or modified during this project.

Consumables
Std Bolt on fitting modified for KH Work
3 Way Tee Bolt on fitting modified for KH Work
Equipment Package
Jackscrew Machine
Plugging head
TD-116 Tapping Machine
TD-116 Machine Extender
Kit, Bypass Piping
Shortcutt valve 2"-Keyhole special with QC
2" Hole Saw
2" Holder Pilot
2" Plugholder f/T-101
Air Motor (Right angle drive)
Sealing Element for std wall pipe
Tools package
Tool, hole guard and machine guide (f/18" Keyholes)
Tool, Gasket spreader -f/Std Fitting
Tool, Gasket spreader - f/3 way tee
Tool, Fitting Installation
Tool, Fitting Nut starter
Tool, Fitting Nut tightening
Tool, Torque Wrench
Tool, Fitting Pressure tester (Pre-Tap)
Tool, 2" Shortcutt Valve Install'n
Tool, Completion Cap
Tool, Valve equalization and Machine de-coupler.
Tool, Shortcutt valve open/close
Tool, Measuring
Piping, Blowdown w/quick coupling adapter

Table 1

REFERENCES

1. ***Keyhole Technology***: Jarnecke, 2001
http://www.gtiservices.org/keyhole/~3944251.pdf#search='gti%20keyhole '

2. ***Request for proposal, Research Opportunity notice***, Jarnecke, May 30,2003
http://www.gtiservices.org/keyhole/Keyhole%20RFP%20Service%20Installation%202.pdf#search='gti%20keyhole'

3. ***A Better Way to Implement New Technologies***.from Gas Operations News-a GTI Publication, June 2004, Vol. 1, Number 1
http://www.gri.org/webroot/downloads/en/1ResearchCap/1_1GasOps/gas_ops_news_June04.pdf#search='gti%20keyhole'

4. ***Keyholing, Attracting Renewed Attention***, by J. Griffin Underground Construction magazine, Feb 2004 see website:
http://www.undergroundinfo.com/uceditorialarchive/Feb04/febkeyhole.pdf#search='gti%20keyhole'

5. ***Small Circular Keyholes cut Costs***, Jarnecke, Gas Industries article, Nov 2003, see website:
http://www.gasindustries.com/articles/nov03c.htm

6. ***KeyholeTtechnology links*** http://www.gtiservices.org/keyhole/links.html

7. ***TDW Website*** http://www.tdwilliamson.com/

8. ***JCM Industries*** *http://www.jcmindustries.com*

Fiber-Reinforced Composite Sandwich Technology: Expanding the Application Envelope for Cured-In-Place-Pipe Products

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Abstract

Rehabilitation of medium and large diameter gravity pipes using traditional cured-in-place technologies often requires heavy, thick-walled liners, creating challenges for transportation and installation. A fiber-reinforced version of industry-proven CIPP technology has been developed as an innovative and reliable solution to these challenges.

Using laminated composite design methodology, similar to that used in the aerospace and sporting goods industries, design engineers incorporate glass and carbon reinforcing fibers into the wall of a cured-in-place pipe. Optimal orientation of the reinforcing fibers allows the design engineer to achieve higher stiffness and strength than with traditional CIPP materials. The resulting laminated composite pipe provides full structural performance with approximately 40% less wall thickness than conventional CIPP materials, delivering the added advantage of increased flow area in the finished product.

The increased physical properties of this composite material provide even more advantage when rehabilitating noncircular host pipes with large radii or flat profiles. This new product expands the technical envelope for cured-in-place pipe rehabilitation beyond traditional application boundaries.

This paper discusses the design theory and product features of the composite pipe product. It also details industry standards testing and the application envelope for this technology. In addition, case studies of successful applications will be presented.

Introduction

Cured-in-place pipe (CIPP) has proven itself over the past 35 years as a reliable, cost effective method for rehabilitating underground pipes. Pipe of almost any size can be rehabilitated using this trenchless technology.

Large diameter, thick-walled, cured-in-place pipes (CIPP), however, present a unique set of engineering and logistics challenges that increase the risks associated with rehabilitating large underground pipes. These challenges begin to manifest themselves at the first stage of the project, when the large CIPP tubes are fabricated in a manufacturing facility, and become increasingly daunting as the project continues through the tube impregnation and installation stages. Fiber-reinforced composite technology can be used to reduce the wall thickness of large diameter CIPP products, thereby reducing the weight and associated risk of rehabilitating large diameter pipes.

The cured-in-place pipe rehabilitation process: The CIPP process begins with the fabrication of a "dry" tube. As shown in Figure 1, these dry tubes are typically constructed by sewing together layers of absorbent fabrics to form a lining. Needle-punched polyester felt is a common fabric used in the construction of CIPP products. In the most common configuration, the outermost layer of the tube is formed using a fabric that has been coated on one side with a chemical resistant, water impermeable polymer thereby creating a leak-tight tube.

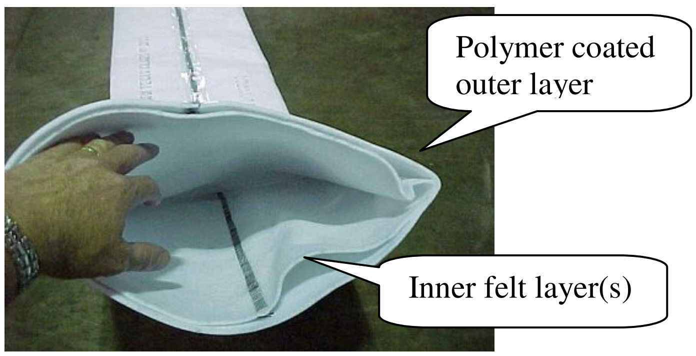

Figure 1

In the case of large diameter pipes, the layers of fabric become so wide that special equipment and processes must be used to handle them. For example, the fabric layers required to construct a tube for rehabilitating a 96 inch diameter pipe, as shown in Figure 2, are approximately 25 feet wide. The extraordinary size and weight of the finished dry tubes requires manufacturing equipment specifically designed to handle these loads.

Figure 2

The next step in the CIPP process is to saturate, or impregnate, the dry tube with resin. In this process, resin is pumped into one end of the dry tube which then passes through a set of rollers which squeeze out excess resin and establish the thickness of the final tube. The saturated tube is then typically loaded into a refrigerated truck for transport to the installation site. The chemical resistance, mechanical properties and ubiquitousness of thermosetting polyester resins make them the material of choice for CIPP products in most applications. The amount of resin required to impregnate a large CIPP tube, however can cause significant logistical problems.

For example, the amount of resin required to impregnate 500 feet of 72 inch diameter by 36mm thick CIPP tube is approximately 90,000 lbs. As pipe size increases, resin storage capacity and pumping time become more significant considerations. Conveying equipment must be designed to support and pull the increased weight associated with these large tubes. It is easy to understand why resin-impregnation of large tubes requires significantly more time than usual; in some cases the process may span several days. The extended impregnation time consumes a portion of the pot life of the catalyzed thermosetting resin, increasing the risk that the resin will begin to cure before intended. In some cases the CIPP tubes become so large that they are too heavy to be transported by truck, and require an onsite impregnation process. Portable pumps and conveyors must setup at the installation site and pump the resin directly from the tank trucks. The level of difficulty and risk associated with the onsite impregnation process is much higher than normal. In most cases an onsite impregnation requires the installation process to be conducted concurrently which in effect doubles the opportunity for process issues to occur, thereby increasing the project risk.

Installation and curing are the final two steps of a CIPP rehabilitation project. Often, the weight associated with large thick-walled cured-in-place tubes requires larger equipment, such as that shown in Figure 3, to be procured for lifting and moving the tube on the jobsite. Handling techniques that work well with smaller CIPP tubes may damage large tubes, simply due to the tremendous forces required to move such a tube into position for installation. Additional safety precautions must be considered based on the weight being handled.

Figure 3

The installation process for CIPP tubes involves inflating the tubing into the host pipe via means of water or air pressure. The volume of fluid required to invert a large tube is obviously much greater than usual. Using the example of 500 feet of 72 inch pipe, more than 100,000 gallons of water is needed to fill the cured-in-place tube. In this situation fluid flow rate becomes a governing factor on installation time. Longer installation time is not only more costly from an installation labor standpoint, but also consumes more of the remaining pot life of the resin.

The amount of energy required to cure CIPP is directly related to the mass of the resin in the pipe, which is a function of the diameter and wall thickness. It stands to reason that large, thick-walled cured-in-place pipes require more energy to cure. When water is used to inflate CIPP, it is common practice to also use the water as the curing media. Typically, a boiler truck is used to circulate and heat the water to temperatures that initiate the chemical curing process of the resin. Once again, the large diameter CIPP process is extraordinary because of the amount of energy required to heat the large volumes of water associated with these pipes. Using the example of 500 feet of 72 inch pipe, over one hundred million BTUs would be needed just to heat the water to temperature required for curing.

Now that some of the challenges and risks associated with large diameter CIPP projects have been identified, the discussion shifts to the role fiber-reinforced composite technology plays in reducing them. The improved physical properties of a fiber-reinforced composite can be applied to cured-in-place pipe to reduce the wall thickness required to withstand a given set of loads. Cured-in-place pipes with thinner walls require less resin and weigh less. Thinner tubes also reduce handling difficulties during the dry tube manufacturing process. Further, the challenges during impregnation are reduced due to the smaller volume of resin to be processed and the reduced weight to be handled in the impregnation facility. The reduced wall thickness and weight of fiber-reinforced CIPP tubes becomes even more important during the transportation and installation process. In some instances, avoiding the increased logistical difficulties and risks associated with an onsite impregnation, may be possible by reducing the wall thickness and weight of the tube to one that can be transported by truck. Lighter tubes reduce installation equipment requirements and lower many of the aforementioned risks associated with the installation process.

How does fiber-reinforced composite technology reduce the wall thickness required for cured-in-place pipes? In the United States, the design of cured-in-place pipes is governed by ASTM standards such as ***ASTM F1216*** *Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin-Impregnated Tube* (2006). Equation number X1.3 of that standard (p. 484) defines the relationship between the loads imposed on a cured-in-place pipe used to rehabilitate a host pipe in fully deteriorated condition, and the wall thickness of the cured-in-place pipe required to withstand these loads without collapsing. As depicted in Figure 4, fully deteriorated load conditions require the CIPP to be designed to support all of the loads applied to the pipe; hydraulic, soil and live loads.

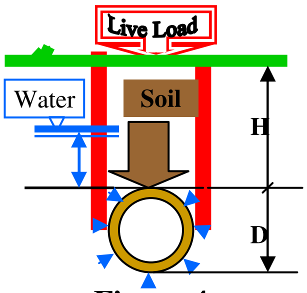

Figure 4

Equation number X1.3 can be rearranged to into the following form:

$$\mathbf{t} = 0.721D\left(\frac{(Nq_t/C)^2}{\mathbf{E_L} R_w B'E'}\right)^{1/3}$$

Where:
t = the wall thickness of the CIPP
E_L = the long-term modulus of elasticity of the CIPP materials
D = the diameter of the host pipe
N = the factor of safety
q_t = the total external pressure on the pipe
C = the ovality reduction factor
R_W = water buoyancy factor
B’ = coefficient of elastic support
E’ = modulus of soil reaction

The geometry of the host pipe, the loads imposed on it, and the factor of safety, are constant for a given design problem. If we remove those terms from the equation, it becomes obvious that E_L (the long-term modulus of elasticity of the pipe wall material) is the only variable that affects the wall thickness of the CIPP. The stiffness of the pipe wall determines the thickness required.

$$\mathbf{t} = \left(\frac{1}{\mathbf{E_L}}\right)^{1/3}$$

Fiber-reinforced sandwich composite technology provides the means to dramatically increase the long-term modulus of CIPP materials and reduce the wall thickness of cured-in-place pipes.

Fiber-reinforced sandwich composites are commonly used in industries such as sporting goods and aerospace to produce structures with high stiffness and low weight. Familiar examples, as shown in Figure 5, include skis and snowboards, military aircraft, and sailboats.

Figure 5

These high performance structures utilize laminated beams that have been engineered to take advantage of the material properties offered by fiber-reinforced plastics. The stiffness of a laminated, or sandwich, beam is determined by the material properties and second moment of inertia of each layer (area times the square of the distance from the neutral plane). As shown in Figure 6, a sandwich composite beam is constructed by bonding a layer of very stiff material to each side of a “core” layer with relatively low material properties.

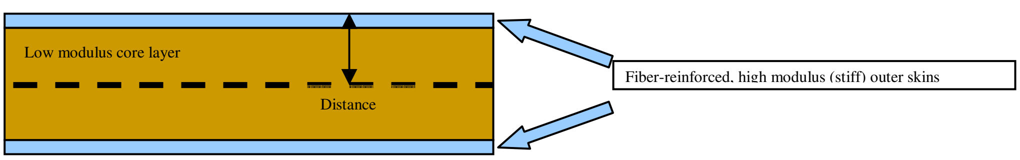

Figure 6

In cured-in-place pipes, the core material is most commonly polyester resin and felt, with a flexural modulus of about 400,000 psi. The high modulus skin layers of a cured-in-place pipe are produced by incorporating reinforcing fiber into the polymer matrix. Glass or carbon fibers are commonly used as reinforcement in sandwich composite beams. By way of comparison, a skin-layer reinforced with glass fiber may have a flexural modulus as high as 10 million psi, and one with carbon fiber may have 20 million psi. In practice, the optimum amount of fiber is designed into the composite beam to achieve the stiffness required.

The stiffness and strength of a fiber-reinforced sandwich composite beam can be calculated using the classical laminated plate theory. This theory makes use of tensors to mathematically describe the contribution of each lamina to the overall properties of the beam. As shown in Figure 7, the modulus and thickness of each layer must be considered. This type of analysis provides the means to optimize the design of the fiber-reinforced sandwich structures for a given application.

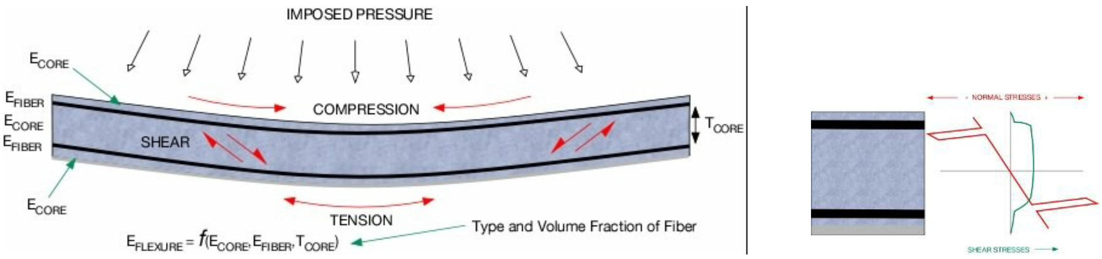

Figure 7

The loads exerted on a cured-in-place pipe are normal to the centerline of the pipe. As a result, the optimum orientation of fiber to resist these loads is that of rings. Two layers of these fiber rings form the high modulus skins of a sandwich composite construction.

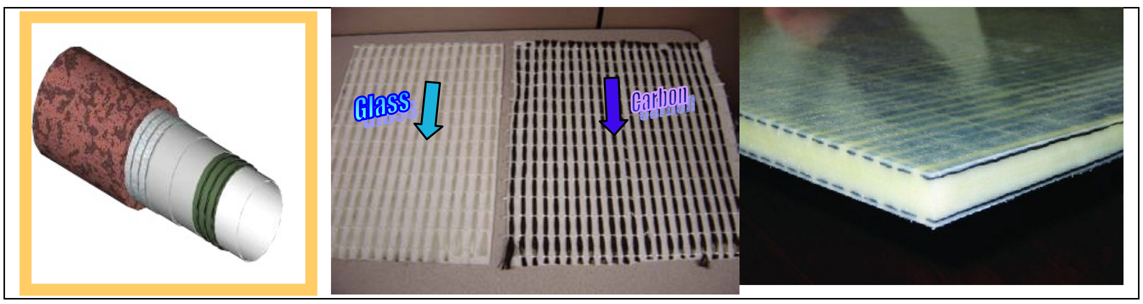

Figure 8

One layer of rings is situated close to the surface of the host pipe with the other close to the inner surface of the cured-in-place pipe. One method of incorporating the reinforcing fiber into the cured-in-place tubes is to produce fabrics as shown in Figure 8. These fabrics are then added to the construction of the tubes during the normal manufacturing process.

As previously mentioned, the long-term modulus of elasticity is used to calculate the wall thickness of cured-in-place pipes. The industry-accepted method employed to determine this material property is to conduct flexural creep testing using the procedure described in **ASTM D2990** (*Standard Test Methods for Tensile, Compressive, and Flexural Creep and Creep-Rupture of Plastics*, 2006). As shown in Figure 9, test samples are subjected to a constant load and deflection is measured over a period of 10,000 hours. The amount of deflection is extrapolated to a 50 year value and the corresponding long-term modulus is calculated.

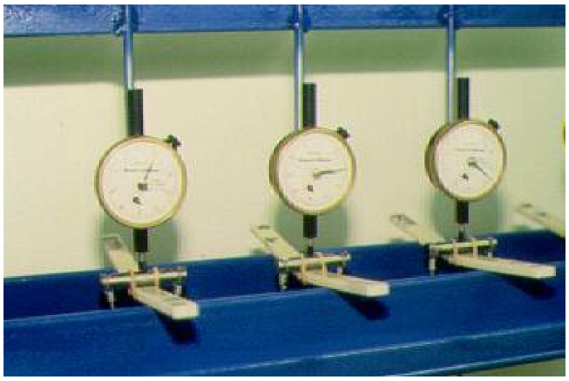

Figure 9

Based on this test, a 50% reduction in the short-term modulus is typically used for traditional monolithic polyester cured-in-place materials. However, fiber-reinforced sandwich composites exhibit less creep than their monolithic counterparts. Independent test results have shown only a 25% reduction in short-term modulus with these materials. So, not only are the fiber-reinforced sandwich composites much stiffer to begin with, but they also retain more of their stiffness throughout their service life.

Let's use the 500 feet of 72" example to compare a cured-in-place pipe product produced with traditional monolithic materials, to one utilizing the benefits of fiber-reinforced sandwich composite materials. Typical inputs to the ASTM equation for a host pipe in a fully deteriorated condition are shown in Table 1.

Table 1

Pipe Diameter	Pipe Length	Pipe Ovality	Soil to Invert	Water to Invert	Soil Modulus	Soil Density	Safety Factor
72 in.	500 ft.	5%	20 ft.	10 ft.	1000 psi	120 lb/ft^3	2

Table 2 outlines the properties of the cured-in-place pipes that would be installed to meet the design criteria set out above.

Table 2

	Traditional CIPP	Glass Composite	Carbon Composite
Modulus (psi)	400,000	1,066,800	1,659,200

E_L Retention Factor	50%	75%	75%
Wall (mm)	36	23	20
Weight (lbs)	89,560	59,850	50,970

In this example, a wall thickness of 36mm would be required to withstand the design loads using a traditional monolithic CIPP design. The total weight of this tube exceeds that which can be transported via truck. A glass fiber reinforced sandwich design reduces the required wall thickness to 23mm, and utilizing carbon fiber further reduces the required wall thickness to 20mm. Both fiber-reinforced sandwich composite designs fall within the acceptable limit for truck transportation. In this example, the additional risk and cost associated with an onsite impregnation process could be avoided by using a fiber-reinforced sandwich composite cured-in-place pipe technology.

The choice of reinforcing fiber is primarily governed by economics. The enhanced stiffness offered by carbon fiber makes it the logical choice when the maximum wall-thickness reduction is the overriding design factor. However, carbon fiber costs about 10 times as much as glass fiber. This expense means that in many cases, a glass fiber reinforced design will provide the most cost effective solution. The manufacturing method described above (for incorporating reinforcing fibers into the cured-in-place pipe wall) makes it easy to use either type of fiber by simply selecting the appropriate fiber-reinforced fabric during the tube assembly process. In fact, a hybrid design with glass on one side of the composite and carbon on the other can easily be produced, if the design requirements call for it.

In order to take advantage of the physical properties of reinforcing fibers in cured-in-place pipes, the composite materials must pass the corrosion requirements set forth in **ASTM F1216** (2006) and **ASTM D5813** (*Standard Specification for Cured-in-Place Thermosetting Resin Sewer Pipe*, 2006). Both carbon fiber and ECR glass fiber perform well in these industry standard corrosion tests.

The advantages of fiber-reinforced sandwich composite CIPP technology recently played an important role in the successful rehabilitation of three large culverts under a heavily trafficked road in Chesterfield County Virginia. The owner, Virginia Department Of Transportation (VDOT), did not want to dig-and-replace the culverts because they would have to close the road for an unacceptable length of time. In addition, seasonal flow requirements eliminated rehabilitation technologies that would reduce the flow area of the culverts, so CIPP was the logical choice of rehabilitation technology. However, the design loads on the 87" by 63" arch-shaped corrugated metal pipes shown in Figure 10 required CIPP tubes with a wall thickness of 61.5mm.

Figure 10

All of the process and logistics challenges associated with large diameter CIPP installations discussed in this paper came into play. By using fiber-reinforced sandwich composite technology, the wall thickness and weight of the CIPP tubes was reduced, thereby lowering the risks and difficulties associated with this job. Table 3 illustrates the comparison between CIPP designs using traditional versus fiber-reinforced sandwich composite materials for this rehabilitation job.

Table 3

	Traditional CIPP	Carbon Fiber Sandwich Composite
Modulus (psi)	400,000	1,163,000
E_L Retention Factor	50%	75%
Wall (mm)	61.5	38
Weight (lbs)	46,300	29,300

Figure 11

As shown in Figure 11, the fiber-reinforced CIPP tubes were fabricated and impregnated with polyester resin using standard large diameter manufacturing processes and equipment. Upon arrival at the jobsite, the CIPP tubes were pulled into place, illustrated Figure 12, and inflated with air. Steam was then used to heat and cure the resin, as in Figure 13.

Figure 12

The steam-cure process, provided several advantages over a water-cure process on this installation. This job would have required the use of 45,000 gallons of curing water. About 7 hrs would be required to raise the temperature of that much curing water from 50 to 180°F (10 to 82° C) with a 200hp boiler. The water would also have to be cooled for several hours before it could be discharged after the CIPP was cured. Using the steam cure process removed about 16 hours from the overall cure schedule. The culverts being rehabilitated carried a creek under the two-lane road. The creek was home to numerous fish, frogs, eels and snakes. Although they never really thanked us, I'm sure they appreciated the fact that by using the steam cure process we avoided the need to discharge 45,000 gallons of water into their home.

The finished product, as shown in Figures 14 and 15, exceeded the customer's expectations of a fully structural pipeline rehabilitation, with minimum environmental impact and no loss in flow capacity.

Figure 13

Figure 14

Figure 15

Fiber-reinforced composite technology has been proven as an effective method to reduce the wall thickness of large diameter CIPP products, reducing both the weight and risk associated with rehabilitation of large diameter pipes. Projects that were previously considered to be outside the technical envelope of the CIPP process, can be engineered with confidence using fiber-reinforced sandwich composite designs. CIPP rehabilitation joins the list of markets that will benefit from future advancements in the science of fiber-reinforced technology. As is the case with wind turbine blades and sports equipment, cured-in-place pipes will take advantage of fibers that are engineered to be stronger, stiffer and more cost effective.

References:

ASTM International. 2006. ASTM D2990: Standard Test Methods for Tensile, Compressive, and Flexural Creep and Creep-Rupture of Plastics. *ASTM Standards*, 202-220.

ASTM International. 2006. ASTM D5813: Standard Specification for Cured-Place Thermosetting Resin Sewer Pipe. *ASTM Standards Related to Trenchless Technology*, 390-393.

ASTM International. 2006. ASTM F1216: Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Invesion and Curing of a Resin-Impregnated Tube. *ASTM Standards Related to Trenchless Technolog*y, 479-485.

Lessons from the Investigation of the Failure of a Water Main Buried Next to 128,000 Volt Electrical Cable

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Abstract

On January 21, 2004, a water main burst spilling over a million liters of water causing multi-million dollar damage. This incident affected the lives of a community significantly forever. The pipeline was directly underneath heavy traffic and was subject to frost loads during its life. This water pipe and a 128,000 volt cable installed in a duct shared a trench and the set back distance was a few centimeters. There was high water table in primarily glacial till and the pipe was made of PVC. The author served as an expert witness for the electric utility performing an independent investigation of the causes of the failure. While sharing rights of way and pipelines for multiple functions is an attractive proposition, the standard of care required of us engineers are more stringent compared to normal utility design, construction, and operation. There were many things wrong with the design, pipe material selection, construction, inspection, operation, and maintenance. This paper documents the lessons learnt for the benefit of pipeline engineers.

Background

On January 21, 2004 a water main owned and operated by a Water Utility failed and caused significant damage to properties and people occupying a housing complex. The author was retained by the regional electric utility during May-November 2004, to perform an independent investigation and to determine the causes of the pipeline failure. The author completed his preliminary investigation and produced a report dated November 30, 2004 for the review of the legal department of the electric utility. The author also requested some additional documents that were needed to complete his investigation to produce a final report. Although the people and their insurance companies sued the design engineering firm, the water utility, the city, the contractor, the developer of the housing complex, and the owner of the housing complex, who they thought were responsible for the damages, they spared naming the electric utility till the last day the court allowed amendments to their original pleadings against other defendants. On December 27, 2005 the author was contacted by the law firm representing the electric utility providing additional data and asking the author to proceed with the completion of the final report. This final report was completed during January-July, 2006 and the findings were reported to this law firm. The author is continuing his work for this law firm. The law firm and the legal department of the electric utility were kind enough to grant permission to the author to share the content of this investigation in general terms at the ASCE 2007 pipeline conference. There were many lessons from this litigation that the author felt were worth sharing with the engineers practicing in the highly specialized discipline of pipe infrastructure for water and wastewater.

Analysis of the Ground Conditions

U.S. Department of Agriculture's Soil Conservation Survey maps for the County were published in 1971 and have been available from their local office in the city for a nominal cost of $ 22 for any member of the public who needed these. The housing complex can be located in these soil survey maps using either the Rectangular or Government Survey System. The land is denoted as North East ¼ of South East ¼ of Section xx in TyN- RzE. Although these soil survey maps were primarily intended for crop cultivation functions and no major engineering decisions could be based on these, the parties involved in this dispute still could have checked readily these maps before any buildings were erected on this site and they would have found the descriptions summarized in Table 1.

In addition, three information circulars and the 7.5 minute Quad maps by the State Geological Survey were reviewed carefully. These publications have been in existence for as little as $ 5 per copy since 1975. These also have been available to the public well before the housing complex was designed and constructed. In these publications, the ground conditions are labeled as those with extremely high yields of groundwater, shallow groundwater tables close to the natural ground elevations in many locations, and coarse grained sands and gravels allowing the frost depth penetration to be deep.

The subsurface exploration report completed on January 16, 1997 for the housing complex by the local geotechnical firm has done a fine job of documenting the soil conditions. The boring logs taken during the geotechnical investigation and laboratory testing cautioned about excessive groundwater, poor soil conditions, and the presence of mostly sands and gravels in Borings B-5 and B-6 that are the closest to the location where the water main failed on January 21, 2004. Despite the availability of this soil exploration report and the expert guidance of these soil engineers, the design engineering firm, the developer, the contractor, and the water utility never factored these findings in their engineering design of the buildings, location of the underground garage, review and approval of the design, and construction of the water main, respectively.

Table 1: **Soil survey maps data for NE 1/4 of SE 1/4 of Sect x of TyN- RzE**

FsB-fox silt loam, 2 to 6 percent slopes, p 74, sand and gravel, potential frost action-moderate, water table depth> 6ft, cut banks caving severe

HmB-hochheim loam, 2 to 6 percent slopes, p 78, gravelly sand-moderate, >6 ft,

HmC2-hochheim loam, 6 to 12 percent slopes, eroded, p 79, gravelly sand-moderate, >6 ft

HoD3-hocheim soils, 12 to 20 percent slopes, severely eroded, p80, gravelly sand-moderate, >6 ft

Pa-palms muck, p 94, gravelly sandy loam-high, 1 ft, severe ponding of water in basements

Ph-pella silt loam, p 94, sandy loam to silty loam-high, 0.5 to 1 ft, severe ponding of water in basements

Sm-sebewa silt loam, p 99, gravelly sand-high, 1 ft, severe ponding of water in basements

It is ironic that portions of the water main, storm sewer, and sanitary sewer had already been designed and constructed even before this subsurface exploration report was made available making one wonder what the purpose of the developer and his engineer of record obtaining this geotechnical engineering report. This report was obtained after construction started and no one paid any attention to the content of this report and there was never a soil engineer present at the job site during construction despite this requirement was stated in the subsurface exploration report by the local geotechnical engineering firm.

Analysis of the Pipe Coupling

On January 26, 2006 when plaintiff's experts dismantled and inspected the pipe coupling that came from the failed 45 degree bend of the water main that burst the author was able to document in detail, the physical conditions of the coupling, bolts, nuts, and the gasket. The condition of the ends of the six bolts where the nuts were threaded was carefully inspected by the author. There was absolutely no damage to the ends of any of the 6 bolts or to the integrity of the threads as shown in these photographs. The scratch marks on the coupling itself the author saw were nothing but what appeared to be those made by the hand tools used by the repair crew of the Water Utility when they were taking this coupling out of the trench with the failed section of the pipe measuring 93 inches, on January 21, 2004.

The entire assembly of bolt # 6, that was removed last from the coupling with the nut threaded back on the stem of the bolt that the engineering manager of the water utility had claimed as the bolt that had mushroomed due to a heavy blow during construction of the 128 kv electrical lateral was absent of any mushrooming. This bolt needed no extra effort on the part of the employee who was removing the nut from the bolt and threading it back in front of all of us who were there that morning for the inspection on January 26, 2006. When the author inspected all 6 bolts for mushrooming thoroughly, he could not find any such defects on any of the six bolts. Bolt # 6 in the author's engineering assessment as a matter of fact looked better than the other 5 bolts. In fact, if the author was not told repeatedly by the attorney defending the water utility during the dismantling of the first 5 bolts willingly but with so much reluctance on his part only for this bolt # 6, the author would have never known from his own inspection that bolt # 6 indeed was the one that was asserted by this attorney as mushroomed.

The nut from a bolt that had received a heavy blow from a piece of construction equipment and had mushroomed would not be easy to unthread and rethread but in the case of bolt # 6, the unthreading and rethreading of the nut went without any problems whatsoever. The author inspected the end of this bolt # 6 thoroughly by running his index finger right along the periphery of the end of the bolt looking for even a minor blemish but found none and the bolt appeared to be in fine condition to be able to accept the nut back on its stem readily. The gasket felt rather brittle upon inspection with minor flaking and ripping due to the coupling having been exposed to unusually low temperatures during the time it was in service. The gasket also would not bounce from the table at all when it was dropped deliberately by the author, indicating that the value of the coefficient of restitution of this rubber had gone down due to the harsh temperatures it had faced during its short life. The author is also wondering whether the six bolts were tightened by the contractor at the recommended torque by the manufacturer in the right sequence. There appears to be no record of this at all in the files.

Analysis of the Choice of Pipe Material and Design

The original design submittal for this pipeline was of all ductile iron, but somehow some of the pipe got switched to cheaper C900 PVC pipe by the contractor and no application was expected of either the design engineer or the contractor by the Water Utility about the switch and no approval was granted by the Water Utility. It appears that the inspector appointed by the Water Utility was not present on the site to object to the use of PVC pipe although this is a poor choice of pipe material, given many water utilities in cold weather conditions have had problems with this pipe material when made with large amount of filler. Loss of impact strength of PVC in cold temperatures as had been thoroughly documented in many investigations by the pipe industry, for example Lamson & Sessions (2006), loss of tensile strength and modulus of elasticity when excessive fillers are used in the formulation of the composition of the resin need to be factored in the evaluation of the suitability of this pipe material. The pipe was never designed, never specified properly, never constructed adequately, did not have either an adequate soil cover or insulation for protection from the frost, and was never inspected with care. Although the Water Utility's own specifications dated February 27, 1998 required the use of only ductile iron pipe with a minimum cover of 1.83 m(6 feet), this was never enforced on this pipeline by the Water Utility.

Despite the assertion by the water utility that the inclusion of pipeline grade in the design submittals was not a requirement of the Water Utility in 1998, Water Utility's specifications dated February 27, 1998 reads "Before a water main can be installed in a street or (sic) easement, permanent grades must be approved and certified by the city engineer or the owner's registered engineer where the main is to be located within an easement." It is impossible for the Water Utility to expect a record of the final grade filed with their office unless a profile is drawn along with the plan of the water main. Therefore a reasonable conclusion to draw is that the Water Utility always had a requirement of applications for design and construction of water mains needing to have plans and profiles in the submittals. The specifications required "trench bottom must be level and free of large rocks….." but the photographs taken on the day of the repair on January 21, 2004 indicate that there were large rocks in the trench left behind by the contractor and the inspector appointed by the Water Utility never enforced the water utility's specifications. In section 1.18 of the specifications, records of the following:

1) date installed
2) pipe or fitting type
3) nominal length
4) laying length
5) strapping or blocking material used

were to be turned in to the Water Utility's distribution manager at the close of each day. Nevertheless, this requirement was never enforced and no such records exist in the offices of the Water Utility. Furthermore, in Section 1.19 of the specifications it states "All work shall be inspected periodically by the technical services manager or his representative, or distribution system manager. For work being done by a contractor, a resident Water Utility inspector will be present during all construction activities to insure that job specifications are followed." Again, none of these requirements in the specifications were enforced during installation of this pipeline. In Appendix A of the specifications titled "Standards for water main line and grade staking, it states that "The petitioner shall, through his registered land surveyor or professional engineer, provide line and grade stakes, upon request of the Water Utility, in accordance with the Water Utility standards." This was also ignored for this project by the design engineer and the Water Utility.

It is also rather interesting that the design engineer had in their files on this project, the drawings they picked up from the City on the street improvements right along South East Avenue. These drawings on a state DOT project were approved and accepted by the city engineer on March 7, 1995. These 92 drawings included plans and profiles for the pipes installed, somewhat ahead of the time the engineering manager of the water utility asserts that there was never a profile requirement within the City. It is even more interesting that these plans and profiles mention three times the words "insulate the water main" along with a notation in the section titled bill of quantities for a total of 14.9 sq m (160 sq ft) of 100 mm (4 inch) thick polystyrene insulation boards to be used on this project.

The Water Utility, the design engineering firm, the developer, and the contractor, all of them have stated that the pipe design and construction met the requirements of the Water Utility's "Specifications for the Installation of Water Main and Appurtenances" when indeed, they all ignored practically all of these engineering requirements.

Analysis of Construction Procedures

The Water Utility, the design engineering firm, the contractor, and the developer of the housing complex have also stated that the water main they designed, reviewed, approved, and constructed met the requirements of "Standard Specifications for Sewer and Water Construction in the state." The author has reviewed the Fifth edition dated March 1, 1988 and the Addenda 1 and 2 dated January 2, 1992 and March 1, 1999, respectively in their entirety and found that many of the requirements of line, grade, minimum depth of burial, use of insulation, grade boards or laser beams, buttressing at the bends were never met during construction of this water main.

Analysis of the Pipe Bends

There are 8 bends in this water main and an inspector was employed by the Water Utility to be present in the field during construction and for him to prepare and file inspection notes of what he witnessed on a daily basis.

The Water Utility's inspector's notes indicate that he witnessed the contractor install a concrete block at station 2+60 and he also witnessed the contractor install mega lugs at the 45 degree bend directly north of the bend that failed. Although this inspector made detailed notes about what went on at station 6+63 where the pipe failed, no record of the contractor installing a concrete buttress at this 45 degree bend has ever been made by him. Careful examination of the material and fitting consumption sheets maintained by the contractor that they relied on for getting paid by the developer of the housing complex also has no record of additional mega lugs or use of materials needed to provide concrete buttresses or blocking at remaining bends in this water main.

The most authoritative design guidance for the sizing of the buttress for PVC water mains is "PVC Pipe Design and Installation-M23," published by the American Water Works Association. For a pressure rating of 10.2 bars(150 psi) at a 45 degree bend of the PVC water main when buried in native soils of sands and/or gravels with an angle of internal friction of 30 deg and unit weight of 17.2 kN per cubic m(110 pounds per cubic foot), with a soil cover of 4 to 8 feet, for a soil bearing capacity in the range of 0.47 to 0.71 bars(1,000 to 1,500 pounds per square foot), the weight of the concrete buttress needed would range from 545 to 772 kg(1200 to 1700 pounds), allowing for a test pressure of 13.6 bars(200 psi). However, if one were to not pay attention to the conditions under which the standard buttress dimensions were arrived at in the Water Utility specifications, it would lead to a severe reduction in sizing down to a mere 227 kg(500 pounds) in the weight of the buttress constructed.

In conclusion, the Water Utility's specifications required a concrete thrust block of about 227 kg (500 pounds) in weight for the contractor to install at the 45 degree bend that failed and this was less weight than that required of AWWA specifications. It is also impossible for anyone to remove such a massive mass of concrete weighing 227 kg (500 pounds) when it surrounds the perimeter of the water main in the ground as shown in the Water Utility's specifications without ripping the bend of the pipe.

This author had concluded even in November 2004 upon a careful examination of all of the evidence that the contractor never built a concrete thrust block and this is why the inspector appointed by the Water Utility did not record the construction of a concrete thrust block at the location of the 45 degree bend that failed.

Analysis of the Water Utility's Engineering Manager's Deposition

The author attended a meeting with the Engineering Manager and his staff in the Water Utility offices on September 28, 2004. The author asked for a copy of the plans and profiles for the water pipe network, pipe sizes, pipe material types, age distribution of the lines, records of other breaks, and copies of certificates filed by the contractor with the Water Utility showing that the contractor met the pipe material specifications and construction specifications. The author also asked for a copy of SCADA records on the day of the pipe failure. The Water Utility's Engineering Manager answered that they have never maintained profiles but they are in the process of going back and building this profile record for the entire network of water pipes in his system and that he would be happy to provide plans and profiles in one month. To this day, the Engineering Manager has not opened his files on these records to the author for inspection and copying, although these are all accessible by any member of the public within the open records law governing a public entity.

The manager in charge of the Water Utility's field operations, answered during this meeting of September 28, 2004 that they had 18 other pipe breaks during the winter season when this pipe failed on January 21, 2004 and that they have never gone back and conducted any form of failure analyses to determine the causes so he had nothing to share with the author. He also said that they had no certificates in their files of contractor complying with material or construction specifications. He also said that the pipe was originally installed under 2.43 m(8 feet) of soil but now had only 0.91 m(3 feet) due to the construction of the basement garage driveway.

The Engineering Manager of the Water Utility was unsure whether they had any SCADA records of the pipe failure for that day. He felt that only 5% of their pipe in the entire network was made of PVC.

When asked of Water Utility's Engineering Manager, "how does cold weather of that nature affect a water main?" a licensed professional engineer in the state answered "Really it doesn't have any effect on—on a water main in my opinion." If one were to examine the state's administrative code Comm Ch. 82.40(**8**) it states "Installation. (a) Frost protection. 1. Adequate measures shall be taken to protect all portions of the water supply system from freezing. All private water mains and water services shall be installed below the predicted depths of frost specified in s. Comm 82.30 (11) (c) 2.d., Figure 82.30-1 and Table 82.30-6, unless other protective measures from freezing are taken. "As a matter of fact this requirement from the State Administrative Code is copied and made part of the Water Utility's specifications for the installation of water main and appurtenances requiring that the water mains shall be buried at least 1.83 m (6 feet) deep past the frost depth. Sands and gravels found from borings B-5 and 6 of the subsurface exploration report by the local geotechnical firm dated January 16, 1997, in Zone A of the State Administrative Code Comm Ch. 82.40(**8**) would require that the water mains be installed at least deeper than 6 feet below grade. When asked "what are those reasons for the 1.83 m(6 feet) below grade requirement?" his answer under oath is" off the top of my head, I wouldn't be able to tell you." When asked "Is there a correlation between cold weather and water main failures?" under oath he answered "As far as my experience has been in the last three years that I've been here, there really isn't a correlation between main breaks. They happen when the failure occurs and that happens throughout the year," when indeed this is outright false given the fact that there were 18 breaks in the January 2004 season alone according to the Water Utility's field services manager, at the meeting with the author and others on September 28, 2004. It's rather ironic when asked "how many times a year pipe breaks occur within your Water Utility?" the Engineering Manager of the Water Utility answered "probably under the average of twenty a year." It is quite clear that if 18 breaks out of the average of 20 breaks per year Water Utility experienced occurred during the winter months, then a gross lack of proper engineering design, operation, and maintenance to cope with cold weather are the primary reasons why the lives of over 65,000 people were disturbed numerous times each year, due to 90% of the water breaks occurring during the winter months. Furthermore, in the author's professional opinion, the Engineering Manager of the Water Utility never conducted either his own failure investigation or retained the services of an outside pipeline expert looking over the pipe failure records to learn some engineering lessons to be able to provide a more reliable water service to the 65,000 plus residents within his jurisdiction.

When asked "As matter of physics though, below 4(sic) degrees, above 0 (sic) centigrade, the colder it gets, the more water expands; correct?" the Engineering Manager of the Water Utility answered under oath "Yes. But in a water-main-type scenario, there is room for it to expand within the water main. When asked again "water is incompressible; right," he answered "Not in a water main system." It is quite clear that the Engineering Manager does not understand even basic principles of how water mains are designed, operated, and maintained and the fact that water indeed is incompressible and this is why we have the problem of water hammer in water mains. When asked "Water is compressible inside the water mains?" his answer was "Well, because there is areas (sic) for it to expand in all our storage facilities." To the contrary, when water expands due to freezing and tries to expand in the axial direction of the pipe, the friction between the inner wall of the pipe and the ice that forms from water would become so great, there would be no movement of water at all into the storage facilities to accommodate the room needed for this expansion that results from water changing into ice by increasing in volume by about 10%. This is why most buried pipe designs assume that the design problem at hand is a plane strain problem, which means that every cross section taken for design calculations along the length of the line at different stations stay the same; therefore, the pipe could be designed adequately by designing a cross section of the pipe wall. As this process continues, the ice will impose loads the water pipe was never deigned for and would cause longitudinal splitting of the pipe due to the hoop stresses in the pipe exceeding the tensile strength of the pipe material, resulting in the fluid gushing out of the failed pipe. This is exactly what happened in this failure on January 21, 2004, and this is why the pipe split over 2.36 m (93 inches) in the longitudinal direction.

When asked, "Do you know who did design that insulation into the repair?" The Engineering Manager of the Water Utility answered "It was- - It wasn't a design, so to say. It was kind of a common agreement between all the people that (sic) were out at the site. Again, the manner in which the Water Utility took its decision on water main operation and maintenance was not based on sound engineering design principles. Instead they took a vote at the repair site among members of the repair crew, the contractor of no experience installing a water main of this size before this project, and the residents of the housing complex on how to repair the water main break so that it does not happen again. In fact, section 82.40 (**8**) of the State Administrative Code has a requirement that the water main shall either be installed below the predicted depths of frost specified in s. Comm 82.30 (**11**) (c) 2.d., Figure 82.30-1 and Table 82.30-6, or adequate insulation materials are used to protect the water main from freezing as required in s. Comm 82.30 (**11**) 3. a. Purely by accident and not by conscious desire to either engineer the repair of the break correctly or meet the legal requirements of protecting the water main from freezing as stated in the State Administrative Code that the engineering team from the Water Utility installed an insulation layer for the first time ever above the water main that was buried well above the frost depth in this location, that was in violation of the State Administrative Code during June 1997- January 2004.

Analysis of Pipe Break Data from Other Water Utilities

It is interesting to review what happened during the same days of that year in other locations around the country. For example, Milwaukee Water Works manages a total of 3136 km (1,960 miles) of distribution piping in their network with an average annual pipe break rate of 700. During January 2004, they had 116 breaks and in particular during January 20-22, they had 37 breaks indicating that the unusually cold weather was a major contributing factor to them having excessive number of breaks on those days.

It is rather interesting to observe the pipe break pattern even in Portland, Oregon on the very days in January, 2004. City of Portland has 3280 km (2,050 miles) of pipe and on the average incurs about 0.11 breaks per mile per year, compared to Milwaukee with 0.36 breaks per mile per year, totaling about 225 breaks per year. And 40 of these breaks in 2004 took place in the middle of January 2004 in Portland indicating that even on the west coast, an unusually cold weather during January 2004 took its toll on the water pipe network. The City of Portland spent a mere $ 136,000 to deal with these 40 breaks, in comparison to a single break of the Water Utility in this dispute on January 21, 2004 costing many members of the public multi-million dollars.

Lessons Learnt

The following lessons can be learnt from this independent investigation:

1. There have been ample soils data in the public domain prior to the design and construction of this water main under the driveway, clearly indicating problems that would surface if proper engineering design and construction precautions are not undertaken. However, it is the author's opinion that the Water Utility, the developer of the housing complex, the contractor, and the design engineering firm never did an adequate due diligence.

2. There is no evidence of any of the bolts of the pipe coupling being either damaged or mushroomed by a heavy blow. And there was never a concrete buttress at this 45 degree bend of sufficient weight to resist the unbalanced forces created by the water pressure. Even if such a buttress was present, it would have surrounded the PVC pipe for it to have worked, and would have been wedged firmly against the native soils. And it is impossible to move such a massive mass of concrete weighing about 227 kg (500 pounds) and even if someone attempted to move this buttress, this would have damaged the PVC pipe and the coupling. Therefore, there is no evidence to support these claims.

3. The choice of the pipe material for the water main is poor and it appears that the contractor never had any experience of installing AWWA C-900 10.2 bars (150 psi) water main prior to this project and thus did not provide adequate care during construction. Neither the Water Utility nor the design engineering firm provided an adequate review, inspection, or QA/QC of the design, specifications, pipe bedding, and the construction of this water main.

4. There is no evidence to support the assertions by the Water Utility, the engineering firm, the developer, and the contractor that they met the requirements of the Water Utility's specifications, AWWA specifications, ASTM

specifications, and Standard Specifications for Sewer and Water Construction in the state. In fact, if the Water Utility had maintained records of the grade of its water mains making the digger's hotline work for other utilities, much of what happened in this accident could have been avoided.

5. As a matter of fact, the Water Utility, the design engineering firm and the other defendants have violated many of the sections of the state's Administrative Code and have compromised the health and welfare of many members of the public.

6. The construction procedures used by the contractor for this water main simply did not meet many of the sections in the Standard Specifications for Sewer and Water Construction in the state.

7. In the author's opinion, the pipeline and the bends were never designed or approved by anyone and the Water Utility staff never provided either an adequate review of the submittal from the developer and/or the design engineering firm or an adequate field inspection of contractor's work.

8. Unusually high number of pipe breaks even without an electrical cable in the same trench in the pipe networks owned by Milwaukee Water Works and the City of Portland, Oregon indicate that the cold weather had a lot to do with these breaks. And when other factors such as reckless management of a water utility that owns the water main, lack of an adequate design and construction are considered, the likelihood of the pipe breaking increases even more and this is why 18 out of the average 20 breaks/annum the Water Utility experienced took place in a few days of January 2004.

9. A single break of the water main owned and operated by the Water Utility on January 21, 2004 resulted in an enormous third party damage and affected the lives of many people in this city. This is due to the Water Utility not knowing something as simple as where their pipelines are buried in the ground and keeping no records essential to owning and operating a community water system. Although the water utility has experienced on the average 20 breaks per year, neither any records were kept of these historical breaks nor causes of these failures were ever investigated toward learning lessons to lower the service disruptions, thirty party damage claims, and impact on people's lives.

10. This water main burst on January 21, 2004 due to a) it being manufactured with an unsuitable pipe material, b) it was never designed, c) it was never constructed properly, d) it was never provided either a deep enough soil cover or an insulation to protect it from the cold weather and intense traffic loads, e) there was never an adequate thrust restraint at the 45 degree bend, and f) most importantly the owner of the pipe has not afforded an appropriate standard of care in designing, reviewing, operating, and maintaining the pipeline putting the health and safety of many members of the public at risk.

11. It was the developer of the housing complex who initiated the design and installation of the water main and an extra duct right in the same trench inviting the electric utility to provide their cable through this second duct. There is nothing wrong with this engineering design and as a matter of fact, the author chairs a working group within IEEE that is developing guidelines on sharing ROWs and underground structures among multiple utilities. He also serves as the American representative on a global working group within Cigre' WG-B1-8 that is completing an engineering guide on sharing tunnels and bridges among multiple utilities. Having an electrical cable close to the water main only helped in this situation by the heat generated from the cable warming the water pipe and the soils around it to delay the burst from occurring sooner.

12. Designing and constructing an underground pipeline does not happen by accident. It requires many years of experience in diverse fields of geotechnical, hydraulic, structural, and pipeline engineering and close attention to the details. Sharing rights of way for multiple utilities pose even more challenges. If the various parties involved in the pipeline project do not recognize these simple facts, the end-product will be a dismal outcome like that described in this paper.

Reference

Lamson & Sessions (2006), "Vylon PVC Gravity Sewer Pipe Installation Guide", p 8.

Appendix - Documents and Evidence Reviewed

1) Various interrogatories, requests for admissions, and production of documents by various parties and the responses.

2) Files maintained by various parties on the design, construction, operation, and maintenance of the housing complex, the waterline, and the electric cable.

3) Transcripts of depositions taken of other experts and the employees of various defendants.

4) Reports of investigation by other experts.

5) Various pleadings, amended complaints, cross-complaints, and the responses.

6) State's Administrative Code.

7) Long term performance prediction for PVC pipes by CSIRO and Iplex for AWWA Research Foundation, 2005.

8) Standard Specifications for Sewer and Water Construction in the state, 1998.

9) Subsurface exploration and foundation evaluation for the proposed housing complex prepared by the local geotechnical engineering firm.

10) Mellburne B. Fielding and Arieh Cohen's paper titled "Predicting Pipeline Frost Load," Journal of American Water Works Association, November 1988.

11) Roy Brander's paper titled "Minimizing failures to PVC water mains," city of Calgary waterworks department.

12) A.Chastain-Howley's paper titled "Transmission main leak: how to reduce the risk of a catastrophic failure," WPRC, forth worth, Texas in Leakage 2005 conference.

13) Brian Boman and Sanjay Shukla's paper titled "materials and installation of delivery pipes for irrigation systems," University of Florida IFAS extension.

14) Ground-Water Resources of the County, U.S. Geological Survey, Information Circular, 1975.

15) Geological topographic maps, 7.5 minute series Quadrangle for the county, 1994.

16) Soil Survey of the County, U.S. Department of Agriculture, First Printed in 1971, Issued 1998.

17) 92 Cut Sheets and As Built from the State's Department of Transportation Plan of Improvement, South East Avenue, Approved and Accepted by the City.

Lessons Learned from Large Diameter Sanitary Sewer Pipe Bursting Project: Conversion of Abandoned Gravity Sewer Main Into Upsized Sanitary Force Main South San Francisco, CA

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ABSTRACT
As an increasing number of large diameter pipe bursting jobs are attempted, more is being learned about the capabilities and potential of pneumatic pipe bursting. The large diameter pipe bursting project in South San Francisco is one of the most challenging and innovative bursting projects ever attempted in North America. Because the project's scope, difficulty, pipe diameter and tooling requirements it is being heralded as a landmark pipe bursting job.

INTRODUCTION
In 1999, the City of South San Francisco was confronted with a Cease and Desist Order from the Regional Water Quality Control Board mandating the elimination of severe wet weather overflows within ten years. The City completed a comprehensive evaluation of their wastewater collection, conveyance, treatment, and disposal system. Locally deficient sewers, undersized pump stations, and hydraulic restrictions in the effluent outfall system were identified as the major contributors to the wet weather overflow problem. A five-phase program was developed to address these conditions.

Resolving the wet weather overflow problem was such a monumental task that many different elements were required. This massive "tool box" of solutions means that every presentation attendee should be able to take away valuable insights to benefit his or her own agency.

PROJECT BACKGROUND
Construction of the first phase of improvements has recently been completed. Recognizing the importance of making significant strides early in the program, the first phase was the largest project in the group. Phase 1 Wet Weather Improvements included:

- Replacement of an existing pump station with a new 45-MGD pump station.
- Construction of a new 49-MGD pump station located alongside an existing pump station.

- Construction of 4,100 linear feet of 36-inch diameter force main (1,880 lf installed by pipe bursting from 27-inch to 36-inch); 6,860 linear feet of 42-inch diameter force main (525 lf installed by microtunneling); and 3,340 linear feet of 8-inch to 16-inch gravity sewer replacement (300 lf installed by horizontal directional drilling at Slope=0.0030).
- Expansion of the effluent pump station to the fullest extent possible without constructing a new facility – 60 MGD.
- Construction of a 7.5 million gallon effluent storage pond.
- Addition of 30 MGD influent pumping capacity.

This paper focuses on the design and construction of the force main installed by pipe bursting.

Design and construction challenges included construction sequencing, constrained sites, coordination with other projects, heavy rain and flooding during construction, soft, corrosive, and contaminated soils, high groundwater, congested utility corridors, heavy traffic, freeway crossings, and property issues. Initially, a route was chosen for the new force main to be constructed by traditional open-cut construction. Soils borings performed for the geotechnical investigation, however, identified a high potential for significant differential settlement and a need for extensive shoring that would include, at a minimum, interlocking sheet piles. The geotechnical engineer warned against constructing the force main by open-cut construction methods.

The design team then considered other alternatives for the force main. One part of the project involved constructing a new gravity sewer to replace a 27-inch diameter VCP sewer. An approach that involved utilizing the abandoned sewer as the host pipe for the new force main was developed. This approach would replace the 27-inch sewer with a 36-inch force main, installed using the pipe bursting process.

PNEUMATIC PIPE BURSTING

The Process. Hydraulic and static pipe bursting equipment is common. However, a majority of pipe bursting done in the United States is done with pneumatic tools. During pneumatic pipe bursting, the pipe bursting tool is guided through a fracturable host pipe by a constant tension winch. As the tool travels through the pipe, its percussive action effectively breaks apart the old pipe and displaces the fragments into the surrounding soil. Depending on the specific situation, the tool is equipped with an expander that displaces the host pipe fragments and makes room for the new pipe. As the tool makes its way through the host pipe, it simultaneously pulls in the new pipe, usually HDPE. See Figures 1 & 2.

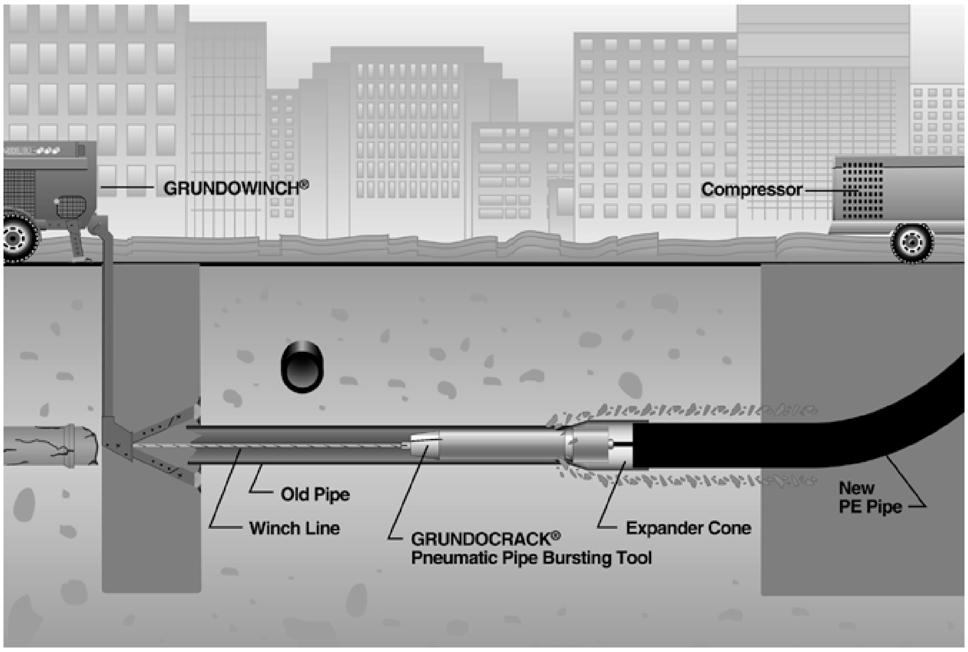

Figure 1 – Pipe Bursting Process

The Tool. With the use of expanders, one tool can be used to burst several different size host pipes and replace them with new HDPE pipes of the same size or larger. Pipe bursting is the only trenchless method of rehabilitation and replacement that allows for the upsizing of the existing pipe.


Figure 2 – New 36-inch O.D. HDPE pipe with 24-inch pneumatic hammer and 42-inch diameter pipe bursting expander ready to launch.

The Expander. Expander and tool configuration can mean the difference between failure and a successful pipe burst. A very common and effective configuration is a pneumatic bursting tool with a rear expander.

Tool and expander selection is affected by various factors. First, is the host pipe fracturable? Fracturable host pipes include concrete, reinforced concrete, clay, cast iron and transite. PVC and ABS plastic pipe offer some bursting potential. With PVC and ABS, special cutting blades are necessary and the length of runs may be reduced. In part, different expander tool configurations are chosen based on the material, size and usage of the host pipe, as well as its depth and profile. Point repairs made to the host pipe may also affect bursting potential.

Second, consideration must be given to the layout of the work-site. Some jobs require both a launch and exit pit. Other jobs are manhole launched and removed. Still other jobs burst from an launch pit to a manhole. The tool is then reversed out, through the newly installed product pipe. This eliminates the need for an exit pit.

Third is the required burst length. In most sewer replacement applications, the burst length is usually manhole to manhole. Long bursts with large diameter product pipes, may require bigger tools and the addition of polymer or bentonite.

Fourth is the terrain and soil conditions. Most favorable bursting projects involve pipes that were originally installed by trenching or open cut because the fill material surrounding them is usually conducive to pipe bursting. Some soils, like beach sand will not remain in the expanded state long enough for the installation of new product pipe.

Fifth is the product pipe size. HDPE is the most common new pipe material. Due to the weight of larger diameter HDPE, bentonite is used to reduce friction. Tool and expander configurations are influenced by product pipe size.

Upsizing depends on the soil conditions, as well.Extremely large upsizes in the 120-125% range have been successfully completed. These bursts are categorized as experimental and out of the ordinary. The 25-50% upsize is much more common, but is still challenging. Upsizes between zero and 25% are considered common.

Bentonite Lubrication. Bentonite lubrication is used in various bursting situations. Due to the weight of larger diameter HDPE, bentonite is used to reduce friction. Friction also increases with the depth of the host pipe. Some soils like beach sand will not remain in the expanded state long enough for the installation of new product pipe. Bentonite lubrication is used in these situations to help maintain the annular space created as the tool travels through the host pipe. Bentonite is pumped through a line that runs inside of the product pipe. A manifold installed behind the tool delivers the mixture to the bursting operation. See Figure 3 & 4.

Figure 3 – Typical discharge port for polymer slurry. This assists the new pipe through the sticky Bay Mud during pipe bursting operations. This port is located directly behind the pipe bursting expander.

Figure 4 – 500 Gallon mixing and pumping system to supplies lubrication that assists with pipe installation during pipe bursting operations.

SCOPE OF PROJECT

The pipe bursting portion of the project called for replacement of 1,800 linear feet of existing 27-inch VCP gravity sewer with new 36-inch O.D. SDR 17 HDPE pipe by the pipe bursting methodology. The existing pipe was approximately 15 feet deep.

Via this part of the project, the existing gravity sewer was converted into a sewer force main.

Features of this project include:

- Significant upsize requirement
- Large diameter host pipe
- Geology (Bay Mud) with high groundwater conditions and an aggressive soil
- Traffic Concerns
- Light Industrial area, many small businesses, warehouses, auto body shops, a concrete batch plant, etc.
- Many driveways to deal with
- Significant truck traffic at all hours
- Host pipe contained a large amount of debris that had to be removed before pipe bursting
- Required Contractor Qualifications for pipe bursting work. It was important to the project designer that a well-qualified and experienced contractor be responsible for the pipe bursting work. The large upsize and the large diameter of the new pipe make this project an excellent example for others contemplating similar work.
- Good coordination between the General contractor, the pipe bursting sub contractor, the equipment supplier, the engineer, and the City
- Resolution of technical problems on site
- Pipe bursting equipment package, included a 24-inch Taurus Pipe Bursting Tool with a 42-inch Rear Expander (See Figure 5), an 18-inch Goliath Ramming Tool (See Figure 6) to push the column of pipe, a 500 gallon Grundomudd system to provide polymer fore lubrication of the new pipe during bursting operations. The use of two 20-Ton Grundowinches, in tandem, to provide adequate constant tension force for this large pipe bursting tool. See Figure 7.

Figure 5 – Note the position of the lubrication discharge port. Also shown are the numerous Grade 8 Bolts securing the HDPE pipe to the pipe bursting expander. Observe that the street is open to traffic.

Figure 6 – Setting up the rear dynamic impact pushing tool. An 18-inch diameter pneumatic hammer is setup in a pipe rammer configuration to assist the heavy pipe string along during pipe bursting operations. This added ramming technique improved pipe bursting production over 50% in the Bay mud conditions present. This rear tool was in operation at the same time as the lead tool and lubrication system.

The two pneumatic tools are not synchronized, each operates at 180 blows per minute.

Figure 7 – Dual 20-ton constant tension winches setup over the receiving pit. The added winch tonnage was required because of the overall weight and size of the pipe bursting equipment and the dead weight of the new 36-inch HDPE pipe. Typical winch tension was 30 tons (each winch operating at 15 tons x 2 = 30 tons). Additional tonnage was available as needed up to 40 tons.

- Support equipment, large capacity Air Compressors, Water supply for Mud system,
- Onsite involvement of the principals of the contractor D'arcy & Harty
- Onsite technical assistance provided by equipment manufacturer TT Technologies
- Specification changes for static pipe bursting and then a return to pneumatic pipe bursting after static method proved inadequate in the difficult geo-technical conditions
- Involvement of inspection personnel
- Problems with one business owner; he could not be satisfied (had sued the City during other parts of the project)
- Connection of pipe segments after bursting operations. Down Hole fusing of flanges and closure pieces
- Length of installed segments achieved significantly longer than expected (420 lf vs. 350 lf = 20%) and installation rate (200 lf/hr) was excellent
- Surface heave/settlement were well controlled and within acceptable limits

LESSONS LEARNED and CONCLUSIONS

- Such an aggressive program is expensive. However, despite the program's cost, the City still has one of the lowest sewer user fees in the region.

- Quality pipe materials, quality workmanship, and overall successful pipe bursting projects are attainable and are a benefit to all parties. Key specifications issues and customer acceptance criteria should include:

- Clearly determine what issues or project problems exist and need to be addressed. No single pipe bursting technology will fit all situations. In some cases, compatible systems may provide the solution desired.
- The engineer or specifier must consider all variables in designing the pipe bursting project to ensure that the product or process will solve the existing needs without creating other problems.
- Specifications should clearly outline the project performance requirements and final goals.
- Specifications must accurately define project pipe bursting requirements, Contractor qualifications and required equipment submittals for pre-acceptance and approval by the Project Manager/Engineer.
- A Quality Control and Safety Plan that details performance criteria should be a required submittal by the Contractor.
- Quality assurance requirements, testing, inspections and quality control documents during construction, must be specified, outlining the consequences and penalties.
- Incentives, if any, for completing the project ahead of schedule and under budget must be clearly spelled out within the project specifications.
- Contract completion requirements must be included in the project specifications including applicable testing and inspection, installation and product warrantees.
- Installation warranty provided by the contractor and product warranty from the pipe manufacturer.
- Include periodic inspections of the complete project in conjunction with warranty periods. The longer the warranty, the more project inspections should be required.
- Repair or replacement requirements for defects discovered during warranty inspections must be adequately defined within the project specifications and reiterated in warranty documentation.
- Extended application warrantees will raise contractor risk resulting in higher project costs.
- Well written performance specifications, including QA/QC, testing and inspection requirements during construction will go significantly further in accomplishing quality pipe bursting projects.
- Adequate geotechnical engineering to verify that pipe bursting is the appropriate construction method and to allow contractor to properly select equipment, mobilize forces, and estimate production rates.

- Locate all critical utilities during design so contractor can place pits, identify important crossings, etc.

Lessons Learned – Lining Asbestos Cement Sewer Main

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Abstract

This paper presents information on the investigation and lessons-learned during recent cured-in-place (CIP) lining project in which the host pipe was an asbestos cement (AC) sewer main.

Due to deterioration of the main, swelling of the AC material resulted in reduction of the inside diameter of the pipe. Upon installation and curing of the liner material, wrinkles were noted during the follow-up inspection.

Material testing and inspection were conducted to determine what, if any, impacts the wrinkling would have on the pipe's integrity.

This paper examines the process of installing the CIP liner, examination of the final product and the resulting impacts of the wrinkles.

Background

In December 2005, a cured-in-place lining of a 356mm (14-inch) Asbestos Cement sewer line located in Atlantic Avenue, Hull, Massachusetts. The AC sewer receives flow from two pump station discharge force mains; one located on Valley Beach Road provides a flow of 12.6 l/s (200 gpm), while a second pump station adds 28.3 l/s (450 gpm). An additional domestic flow of approximately 6.3 l/s (100 gpm) also flows into this pipe segment. Length of the lining work was approximately 442 meters (1,450 linear feet). The segment includes 6 manholes, with MH 22 being the upstream structure where the two force mains enter and MH 17 being the downstream chamber. See Figures 1a and 1b.

In the follow-up television inspection, it became apparent that the CIP liner had areas with wrinkles and folds. The length of main of concern is approximately 298 meters (978 linear feet) in length. Several concerns were noted concerning the impacts wrinkles and folds would have on the finished main and they were:

1. Strength of the pipe liner
2. Reduction of hydraulic capacities
3. Potential for forming obstructions to flow

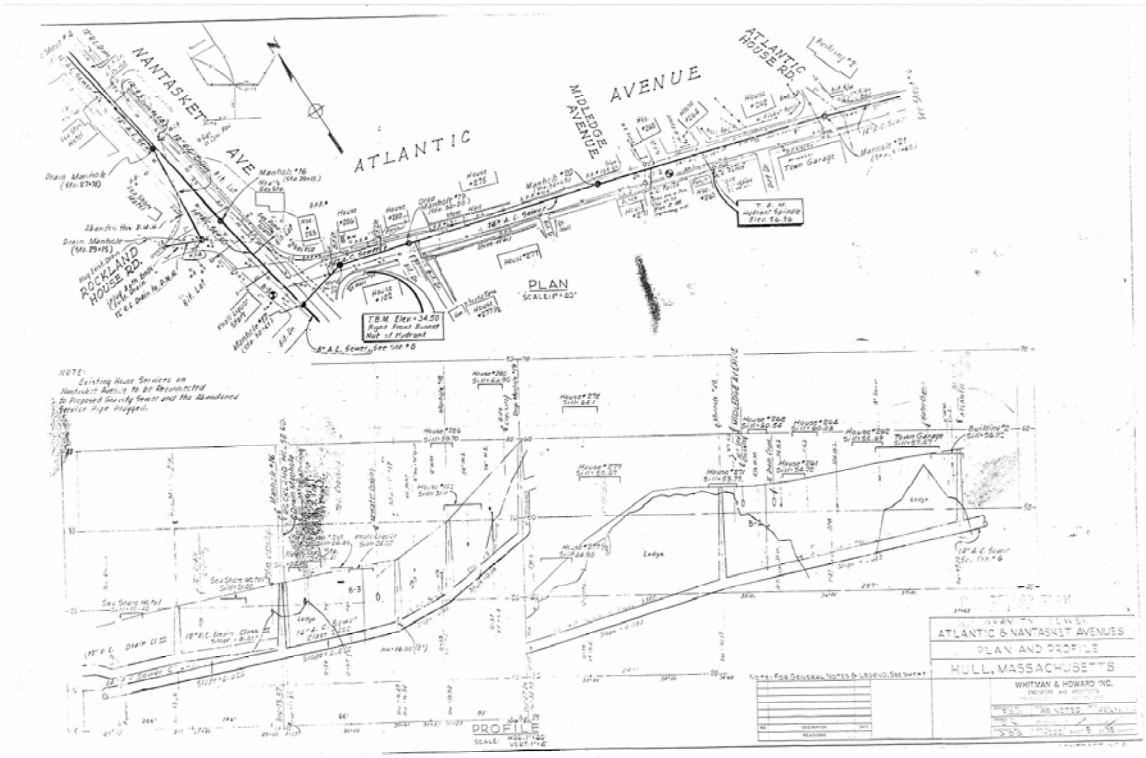

Figure 1a – Atlantic Avenue. 356 mm (14-inch) AC main between MH 17 to MH 21.

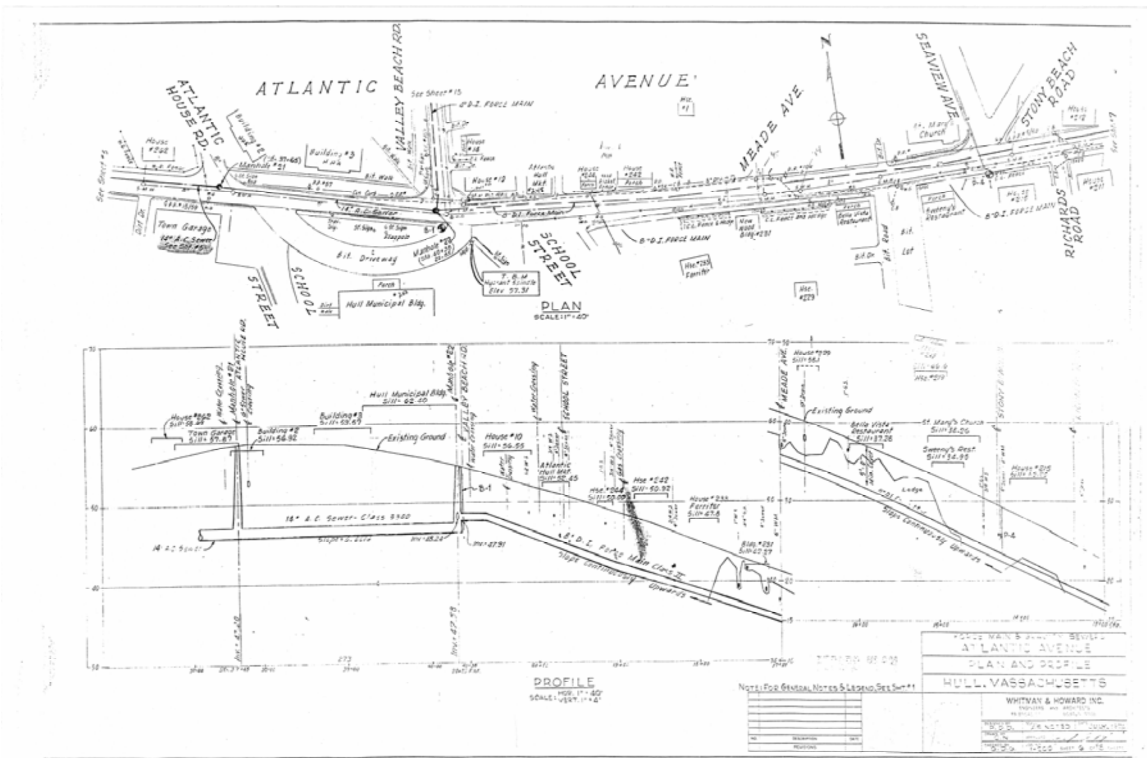

Figure 1b – Atlantic Avenue. 356 mm (14-inch) AC sewer between MH 21 – MH 22. Force mains discharge into the gravity sewer at MH 22.

Wrinkled and partially deformed liners are normally attributed to the liner's mirroring the host pipe sizing and/or inadequate point repair work. The liner generally cannot be perfectly round when the host pipe is deflected or distorted. (Schrock, 2000)

Findings

In order to determine the impacts of the folds on the integrity of the system, a review of test data and videos of the finished pipe was conducted. Also, calculations were made to determine the impacts of the folds on the flow characteristics of the lined pipe. Based on these reviews, it is believed the lined main will provide adequate strength and flow capacity. It is believed that there is not a potential for blockage due to accumulation of debris in the pipe due to the folds.

Strength Issues: The CIP liner was designed to allow the liner to withstand soil, groundwater and traffic loads without benefit of the host pipe. In order to determine if the liner was of adequate design and installation, tests were conducted on the material placed in the 356 mm (14-inch) main by the liner installer

The tests were conducted to determine the flexural properties of the CIP material samples per ASTM D790. The properties determined were Modulus of Elasticity and Flexural Yield Strength.

Modulus of Elasticity is the slope of the initial linear portion of the initial linear portion of the stress-stain curve. It is a constant which is unique for a particular material and is a measure of the stiffness of the material.

Flexural Yield Strength is the maximum flexural strength achieved by the sample during a bending test. It is the point where the material yields with no additional loading placed.

The engineering specifications required the CIP material have a Flexural Yield Strength of 31 MPa (4,500 psi) and a Modulus of Elasticity of 2413 MPa (350,000 psi). Samples were taken from material placed during shot 1 at Manhole 19, shot 2 between MH 19 and MH 17 and Shot 3 at MH 15. The laboratory test data results are given in Table 1. The laboratory test data is attached to this memo.

Table 1 – Flexural Properties Test Results – ASTM D790

Sample Location	Flexural Strength MPa (PSI)	Modulus of Elasticity MPa (PSI)
MH 19	41 (5,896)	3,119 (452,412)
MH19 – MH 17	36 (5,174)	3,310 (480,069)
MH 15	34 (4,994)	2,872 (416,599)

Additional Testing: The liner installer was requested to provide additional pipe strength data on linings installed at other locations which had similar wrinkles. Purpose of this information was to determine if the wrinkles resulted in loss of strength to the pipe.

The liner installer provided a memo on testing they performed on a 432 mm (17-inch) diameter pipe sample. The sample was tested by the installer to evaluate the impact of a wrinkle on the physical integrity of the liner. Two sections of pipe were sampled, one having a wrinkle and one without a wrinkle. The sections were tested for pipe stiffness characteristics in accordance with ASTM D2412 – Determination of External Loading Characteristics of Plastic Pipe by Parallel Plate Loading. Flexural strength properties per ASTM D790 were also determined. The test data results were as shown on Table 2.

Table 2 – Pipe Stiffness Test Results – ASTM D790 and ASTM D2412)

Sample Info.	Flexural Strength MPa (PSI) ASTM D790	Modulus of Elasticity MPa (PSI) ASTM D790	Pipe Stiffness MPa (PSI) (ASTM D2412)	Modulus MPa (PSI) (ASTM D2412)
17-inch without wrinkle	48 (6,998)	4,374 (634,385)	0.17 (24)	3,492 (506,406)
17-inch with wrinkle	48 (7,016)	3,944 (572,114)	0.17 (24)	3,385 (491,018)

The liner manufacturer's conclusion on the test data was that there is no significant difference in the strength characteristics of the wrinkled vs. unwrinkled pipe.

Visual Inspection: Another concern was the condition of the wrinkles themselves. It was not possible to determine from viewing the video tapes if the wrinkles were "solid" or "hollow". In order to make a determination, the liner installer was asked to remove a sample length of the lined pipe so that an examination could be made of the pipe liner. It was also believed that it would be possible to remove a section of the CIP liner from the host pipe and have additional strength tests performed.

Because the host pipe contains asbestos material, it was not possible to safely remove sections of the pipe and have them tested at a laboratory.

A 3 meter (10-foot) length of the 356 mm (14-inch) AC pipe was removed at MH 19 and taken to the subcontractor's facility. On July 12, a visit was made to the subcontractor's facility to examine the removed section of pipe. The examination showed the AC pipe's interior to be swollen and soft. The outer coating appeared to be intact and sound. See Figure 2.

A wrinkle was examined during this visit. The wrinkle was solid and no hollow areas were evident. The wrinkle extended 51mm (2-inches) into the pipe and was approximately 25mm (1-inch) thick. See Figures 3 and 4.

The asbestos material was measured and it varied from a maximum thickness of 48mm (1-7/8-inches) at the 2-o'clock position and was not visible at the 9-O'clock position. The cement outer jacket of the pipe measured 30mm (1-3/16-inches) at 9-O'clock position and 10mm (3/8-inches) at 3-O'clock position. The outside diameter of the pipe was measured to be 413 mm (16-1/4-inches) and the inside diameter was measured to be 340mm (13-3/8-inches).

The CIP liner thickness was measured at 4-points on each end of the sample. The thickness of the CIP liner was measured as 8mm (5/16-inch) at each point. See Figure 5.

Figure 2: Exterior of removed 356mm (14-inch) diameter Asbestos Cement pipe.

Figure 3: Fold on CIP liner inside AC pipe (9 O'clock position).

Figure 4: Close up of CIP liner fold.

Figure 5: End section of AC pipe showing varying material thicknesses.

Hydraulic Issues: A concern regarding the impact of the wrinkles on flow and hydraulic capacity of the pipeline was noted. Several approaches were taken to examine the potential impact to hydraulic capacity of the lined main in Atlantic Avenue.

Research was conducted to determine negative impacts to the hydraulic capacity of a lined pipeline due to wrinkles and fins. One such study (Hart, 2004) was located in which liner folds were examined for detrimental impacts. The study determined that head loss was generated by folds extending from the pipe wall into the flow path. The findings determined that head losses were subject to the depth of the fold intrusion into the flow path and the orientation of the fold to the pipe direction. The study investigated the impact of folds perpendicular to flow In Hull, the folds were primarily parallel to the flow path. The paper concluded that head loss and loss coefficients increased as the ratio of the depth of the fold to pipe diameter increased.

The existing AC pipe had varying inside diameter of approximately 340mm (13-3/8-inches). Published Manning coefficient of friction (n) value for new AC pipe is n=0.010. Based on the age and condition of the unlined AC pipe, the unlined friction value was estimated to be approximately 0.015. The lined pipe has a slightly less inside diameter, 324mm (12-3/4-inches). However, the reduction in diameter is offset by the reduction in the Manning coefficient of friction which, for smooth plastic pipe, is approximately n=0.010. Due to the wrinkling, it was assumed that a slightly higher value of approximately 0.013 could be used for calculation purposes. Assuming all other values such as flow and slope remain constant, the reduction in the n-value results in an approximate increase of 15% in flow capacity.

Based on pump station flows from Valley Beach Road (12.6 l/s) (200 gpm) and Atlantic Avenue (28.3 l/s)(450 gpm) entering MH 22, plus assumed domestic flow from residences along Atlantic Avenue, it was estimated the maximum flow in the Atlantic Avenue gravity main to be 47.3 l/s (750 gpm). The estimated flows in the Atlantic Avenue main were distributed evenly to better estimate the actual conditions. Flow distribution is given in Table 3.

Table 3 – Flow Distribution

Pipeline Segment	Flow Distribution
MH 22 – MH 21	41 l/s (650 gpm)
MH 21 – MH 20	43 l/s (675 gpm)
MH 20 – MH 19	44 l/s (700 gpm)
MH 19 – MH 18	46 l/s (725 gpm)
MH 18 – MH 17	47 l/s (750 gpm)

Using as-built drawings, it was noted the main between MH 21 and MH 22 on Atlantic Avenue is the limiting segment due to a relatively flat slope of 0.0014. The TR-16 Design Guidelines recommend a minimum slope of 0.0017 for a 356mm (14-inch) gravity sewer. This pipe segment can handle approximately 48 l/s (765 gpm) flow at a velocity of 0.52 m/s (1.7 ft/sec) at a full pipe flow depth. The segments between MH 17 and MH 22 have the following flow characteristics are shown in Table 4.

Table 4-Flow Characteristics

Pipeline Segment	Flow Velocity	Pipe Flow Depth
MH 22 – MH 21	0.67 m/s (2.2 ft/sec)	297 mm (11.7 inches)
MH 21 – MH 20	1.46 m/s (4.8 ft/sec)	117 mm (4.6 inches)
MH 20 – MH 19	1.58 m/s (5.2 ft/sec)	114 mm (4.5 inches)
MH 19 – MH 18	1.98 m/s (6.5 ft/sec)	107 mm (4.2 inches)
MH 18 – MH 17	2.32 m/s (7.6 ft/sec)	107 mm (4.2 inches)

From a hydraulic standpoint, the main from MH 22 to MH 17 has sufficient capacity to handle the existing flow rates. Much of the Atlantic Avenue gravity sewer capacity is due to the steep slope in which the mains were installed. One segment, MH 22-MH 21 is the exception however. This segment, where the two pump station force mains discharge into, is relatively flat having a slope of 0.0014.

Also, the observed flows appear to be approximately 4-inches deep in the segment MH 22-MH 21. Using Manning's equation, we estimate the flow at this depth to be about 9.5 l/s (150 gpm). This could represent the flow from the Valley Beach Road Pump Station which is rated at approximately 12.6 l/s (200 gpm). If it is flow from the Atlantic Avenue force main, it is well below the expected pumping rate of 28.4 l/s (450 gpm). It was recommended to the Town that they may want to consider conducting flow tests to determine the exact output of the two pumping stations to determine if adjustments or repairs are required.

In addition to the above flow considerations, concerns were noted regarding possible flow obstructions due to the potential of material being caught on the folds and impeding flows.

Based on the video of the finished pipe, the folds appear to be primarily in the upper sections of the pipe. The majority of the folds are in the 11 O'clock to 2 O'clock position, above the normal flow levels of the main. The location of the folds should not cause major interference with flow in the mains.

Conclusions

Based on the review of test data and observations of taped segments and removed sections of pipe, we concluded that the cured-in-place liner installed in the 356mm (14-inch) sewer main in Atlantic Avenue will provide the Town with satisfactory service. The material strength testing indicates the liner was within the specified strength limits and can maintain the structural integrity of the sewer main.

The wrinkles, being parallel to the flow, should not cause obstructions due to blockages in the line. However, based on calculations, the wrinkles will reduce the flow characteristics of the lined main. Typical Manning n-value for cured-in-place liner is 0.010. Cured-in-place pipe having wrinkles can have n-values in the range of 0.013. Using the higher n-value, our calculations indicate that the Atlantic Avenue gravity sewer is capable of handling the expected flows of 47.3 l/s (750 gpm). This is a conclusion similar to that presented in the previously referenced paper by Hart.

Based on this experience with lining of AC pipe, it is strongly advised that future projects require measuring of the pipe interior dimensions and shape prior to installing of CIP liners. Measurements should be throughout the pipe interior and not just at the end points. The ability of AC pipe to swell results in a smaller than expected diameter and non-circular interior shapes. This reduction in diameter and non-uniform shape are the main causes of the formation of wrinkles and fins in the finished product.

References

Hart, Ellis Dale (2004). "*Head Losses Occurring Through Smooth Pipe Fittings, Liner Folds and Fusion Joints, A Dissertation Presented in Partial Fulfillment of the Requirements for the Degree PhD in Engineering"*, College of Engineering and Science, Louisiana Tech University.

Schrock, P.E., B. Jay (2000). "*Long Term Performance of Pipeline Renewal Liners*", pp 273, Proceedings of ASCE National Conference on Environmental and Pipeline Engineering, Kansas City, MO.

EVALUATION OF THE CORROSIVITY OF HDD DRILLING FLUIDS UTILIZED FOR DUCTILE IRON PIPE INSTALLATIONS

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Many HHD installations utilize specialized drilling fluids, commonly called "drilling muds", which are composed of various organic and inorganic additives, polymers, lubricants, and wetting agents mixed with water. One common component in drill fluids which can be corrosive to metallic structures in certain environments is bentonite clay. This one year study investigates the corrosive effects of various commercial drilling fluids on Ductile Iron Pipe utilizing two widely recognized ASTM corrosion evaluation methods: ASTM G-59 (Standard Test Method for Conducting Potentiodynamic Polarization Resistance Measurements) based on electrochemical properties of metals, and ASTM G-162 (Standard Practice for Evaluating Laboratory Corrosion Tests in Soils) based on coupon weight loss. In addition to comparing the corrosive effects of various drill fluids, five different electrochemical test evaluation methods were compared to results obtained utilizing the weight loss method.

Introduction and Literature Search. Water and wastewater infrastructure deterioration problems and the need for research and funding in these areas have been widely publicized. The American Society of Civil Engineers (ASCE) estimates that the nation's 54,000 drinking water systems and 16,000 wastewater systems face annual funding shortages of $11 billion for water systems and $12 billion for sanitary sewers (Lawson, 2003). As cities and municipalities are faced with an urgent need to replace or rehabilitate their aging utility systems on one hand, and dwindling budgets, tight environmental restrictions and an increased emphasis on user costs on the other hand, they are seeking alternate methods for repairing, replacing, and expanding their underground assets (Allouche and Ariaratnam, 2002). One method of piping replacement which is growing rapidly in the water industry is trenchless installation. Compared to traditional "open cut" trench installations of pipelines, trenchless technology (TT) methods offer advantages in installations such as: 1) protected wetlands or waterways where there are environmental restrictions; 2) burials greater than 15 feet; 3) under utilities, rivers, busy roads, and driveways; 4) areas of contaminated soils; 5) in highly congested/populated areas; 6) under high groundwater conditions; 7) near sensitive structures (buildings, burial grounds, archeological sites, etc.); and 8) in special circumstances (areas where trenching operations would cause major inconveniences to the public such as at hospitals, airports, office buildings, etc., and/or loss of business to merchants).

One such TT method is Horizontal Directional Drilling (HDD), and a growing piping material being utilized in HDD installations is ductile iron pipe (DIP). HDD installation of DIP is a relatively new application of the most popular water piping material utilized in the United States by municipalities and cities for transport of water and wastewater. Since its development in the mid-twentieth century, millions of feet of DIP have been successfully installed in many types of environments (Bonds, et al., 2004; DIPRA, 2003a; Horton, 1988).

Drilling fluids are used during the HDD process to support and lubricate the pipe, to reduce required pulling forces, to clean and cool the cutting bit, to stabilize the bore hole formation, and to remove cuttings from the bore hole (Knight, et al., 2001). The type of drilling fluid used is normally based on the type of soil characteristics with the formulation typically being water based mixtures of bentonite clay or polymers, or both. Rowell (2003, p1507) reported: "Drilling fluids containing bentonite have been tested in accordance with ANSI/AWWA C105/A21.5 (AWWA C105, 1999), Appendix A procedures by U.S. Pipe and have been determined to have a low electrical resistivity value." It is known that in traditional, open-cut construction soil environments, generally the lower the resistivity value the more corrosive the soil is to iron piping. Due to corrosion concerns associated with these low resistivity characteristics of drilling fluids and the lack of corrosion studies, corrosion protection has thus traditionally been recommended for DIP installed in HDD installations where drilling fluids are utilized (U.S. Pipe and Foundry Company, 2004). A literature search on the corrosivity of drilling fluids with respect to DIP, however, failed to reveal any reported research in this area. This absence of information is one of the factors which prompted this study.

Horizontal Directional Drilling (HDD) Installations of Ductile Iron Pipe. The strong demand for trenchless rehabilitation technologies has brought a demand for new installation methodologies and, from many owners, a demand for pipe material alternatives to high density polyethylene (HDPE) pipe (Tenbush and Carpenter, 1998). Many HDD water and wastewater projects were initially installed utilizing plastic pipe. However, as engineers and contractors discover that ductile iron pipe (DIP) with special joints are available for this type of installation method, the use of DIP is growing rapidly. Carpenter reports: "a proven and cost effective pipe, ductile iron has a long history of established performance for the water and sewer markets. Though it has been no secret, recent research has verified that, through the use of flexible restrained joints, ductile iron pipe can be successfully used for HDD and pipebursting." (Carpenter, 2003, p.2)

The two methods of installing restrained joint ductile iron pipe with HDD equipment are either a cartridge method or an assembled line method (Griffin, 2003). In the cartridge method, the joints are connected one at a time in the pit. This is the preferred method where there is a limited right of way or easements. The other method is the assembled line method where the joints for the entire installation length are pre-assembled or "strung-out" along the right-of-way prior to installation. With both installation techniques, pipe sections are pulled into the bore hole with spigots ahead, allowing drilling fluid and slurry of excavated material to flow easily over the smooth bell contour.

In the past there has been the misconception that greater pulling force is necessary for DIP than for a similar size of HDPE and that DIP may separate or pull apart during the installations. According to Griffin (2003) and others, this is inaccurate as reports and case histories have given the indication that pulling forces were actually less for DIP than those required for similar sizes of plastic pipe. A reason for this benefit is that the typical bulk density of an empty iron pipe is normally closer to that of the soil/fluid slurry than it is with lighter pipe materials, and the friction against the walls of the bore hole is less during pull-back. In other words, the DIP are more neutrally buoyant and tend to float through the bore hole near the center rather than float against the top of the hole as do lighter plastic pipe. These studies have also shown that DIP restrained joint designs now available provide restraint and flexibility necessary for HDD installation requirements.

In addition to strength and toughness of DIP, Carpenter and Conner (2003) report that it is also utilized in some areas for its impermeability where distribution lines are installed in areas subject to groundwater contamination by organic compounds. Whereas plastic drinking water pipe walls have been shown to be permeated by such contaminates, DIP are not. These authors report the following advantages of DIP in HDD installations:

- “Standard pressure capabilities up to 350 psi (2.4MPa), or greater upon special request.
- Better distribution of thrust or pulling force around the bell and barrel, and greater allowable pulling forces than other pipe options.
- Liberal, allowable joint deflection with simultaneous joint restraint.
- Quick, easy joint assembly.
- “Cartridge” installation option for limited easements of right-of-way.
- Can be located from surface with commonly used locators.
- Performance capabilities of the pipe are not impacted by elevated temperatures.
- Material strength for handling pull-back and external dead and live loadings.
- Material strength which does not creep or decrease with time.
- Pipe wall impermeable to volatile hydrocarbons, minimizing the potential of water system contamination in the present and future.
- No significant residual bending stresses that could adversely affect future serviceability remain in the pipe after pull-back.
- No significant “recoil” and minimal pipe movement after installation due to thermal expansion and also “Poisson” pressure-testing effects.
- Lack of movement and the inherent strength of ductile iron eliminates potential for shearing of tapped lateral outlets or breakage of pipe due to thermal expansion and contraction.” (Carpenter and Conner, 2003, pp.8-9)

Description of Study. In this study, four commercially available drilling fluids commonly used for HDD installations of DIP were evaluated. These four commonly used fluids were identified based on discussions with drilling fluid manufacturers, DIP manufacturers, and HDD contractors. A fifth drilling fluid mixed with inhibitors was also included in the study. Test method verification included testing of DIP in a control solution of 3.5% (by weight) laboratory grade sodium chloride and distilled water. Previous corrosion studies in salt water have shown the corrosion rate of unprotected DIP in this environment to typically be between three and six mils per year [e.g., 0.003"(0.075mm) to 0.006"(0.15mm) per year] (Gray and Ductile Iron Founders' Society, Inc., 1971, p.320), with a "mil" being defined as 0.001 inch (0.025mm). These environments and their corresponding symbols assigned during the study are as follows:

W	Standard 3.5% salt water (control)
BB	A commercially available Wyoming sodium bentonite
BZ	A commercially available Wyoming sodium bentonite plus corrosion inhibitors
CV	A commercially available polymer emulsion – hydrolyzed polyacrylamide /polyacrylate (PHPA) co-polymer mixed into virgin bentonite
CZ	A commercially available sodium bentonite clay with special polymer additive
P	A commercially available Wyoming bentonite blended with special extenders

All drilling fluid mixes were prepared in strict accordance with the manufacturers instructions utilizing a high speed shear mixer. Concentrations of drilling fluid utilized for the study were the same as those recommended on manufacturer's technical data sheets. Per ASTM G-162, a minimum of 40 cm^3 soil volume is required for every 1 cm^2 of exposed metal surface. To meet these minimum requirements, 8.3 gallons (31.9 L) of mixed drilling fluid or test solution was utilized for every three 4"(100mm) X 4"(100mm) coupons. Over 5,000 pounds (2,270kg) of drilling fluids and control solution were mixed for the study. A total of 60 containers and 180 coupons were utilized for the weight loss evaluation phase of the study.

Corrosivity tests (based on procedures outlined in AWWA C105, Appendix A) were conducted for the 3.5% NaCl control solution, the tap water utilized to mix the drilling fluids, and each of the five drilling fluids evaluated. The resistivity of the drilling fluids varied from 380 ohm-cm (Ω-cm) to 1120 ohm-cm (Ω-cm) and the pH varied from 9.0 to 10.6.

Corrosion testing compared results from a one year weight loss study versus various types of electrochemical tests and methods of analysis. Weight loss measurements were conducted at 1, 3, 6, 9 and 12 months. Electrochemical tests were conducted at 1 week, and at 1, 2, 3, 4, and 6 months. The following two primary ASTM corrosion test standard practices with appropriate modifications for specimen size, electrolyte, and test environment were followed: ASTM G-162 (Standard Practice for Evaluating Laboratory Corrosion Tests in Soils) based on coupon weight loss, and ASTM G-59 (Standard Test Method for Conducting Potentiodynamic Polarization Resistance Measurements) based on electrochemical properties of ductile iron.

Electrochemical corrosion testing was conducted utilizing an EG&G Princeton Applied Research Model 263A Potentiostat / Galvanostat with two phase lock-in amplifier and Softcorr III DC Corrosion Software. Electrochemical test procedures are described in the following section.

Laboratory corrosion testing was conducted on 222 ductile iron coupons cut from a production 8" diameter ductile iron pipe. Prior to coupon preparation, metallurgical and physical strength tests were conducted on the parent pipe in accordance with applicable AWWA (American Water Works Association) standards to verify the pipe met industry requirements. One half of the coupons (111 samples) were uncoated and sandblasted and the other one-half (111 samples) were coated utilizing standard production asphaltic cutback “shopcoat”. The samples utilized for the study are shown in Figure 1.

All weight loss test exposures were conducted in triplicate. Weight loss measurements per ASTM G162 (Corrosion Tests in Soils) were conducted at 1 month, 3 months, 6 months, 9 months, and 12 months. This test procedure therefore required 36 test coupons per test period, per environment evaluated. In addition, three sandblasted coupons and three asphalt coated coupons were evaluated at zero exposure time to determine weight loss attributable to the ASTM G-1 cleaning process. Weight loss measurements were determined to the nearest 0.001 gram.

Figure 1 – Photograph of all DIP coupons utilized for the study.

Electrochemical Testing. Corrosion is an electrochemical process, and as such, can be studied using various electrical testing techniques. The following test methods were utilized as the guides for conducting electrochemical corrosion measurements of the DIP coupons in this study: ASTM G59 (2003) - Standard Test Method for Conducting Potentiodynamic Polarization Resistance Measurements, ASTM G102 (2004) - Standard Practice for Corrosion Rates and Related Information from Electrochemical Measurements, ASTM G106 (2004) - Standard Practice for Verification of Algorithm and Equipment for Electrochemcial Impedance Measurements, and PAR Application Notes (PAR, 2005). A brief description of the various types of electrochemical testing techniques utilized in this study is given in the following paragraphs.

Polarization resistance. As discussed in the above references, standard polarization resistance can be related to the rate of general corrosion for metals which are at or near their corrosion potential commonly referred to as "$\mathbf{E_{corr}}$". This test method utilizes a small potential scan, ΔE(t), applied to a metal sample.

Tafel analysis. A Tafel plot, also known as an E log I curve, is a scan that applies a known current density to obtain a voltage shift from E_{corr}, the freely corroding potential. The measurements are computer controlled and performed at an optimum scan rate represented in millivolts per second over a range of anodic and cathodic potentials. All voltage measurements in this study were conducted relative to a saturated calomel electrode (SCE). Using standard Tafel analysis, the linear portions of the cathodic and anodic legs of the curve are extended with their intersection corresponding to the total corrosion current. Tafel plots can be used to a) estimate anodic and cathodic Tafel slopes, b) estimate corrosion rates, and c) determine what type of geochemical process controls corrosion rates. A schematic of this analysis is shown below in Figure 2.

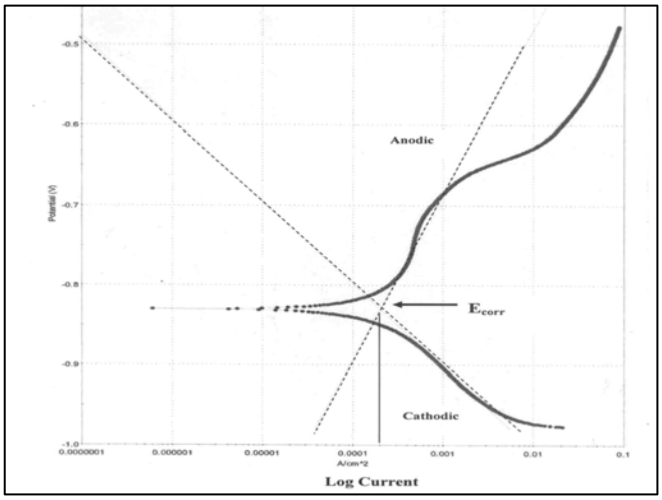


Figure 2 - Typical Tafel Analysis plot showing anodic and cathodic slopes, and the corresponding corrosion current.

Linear polarization. Polarization resistance, R_p, can be determined experimentally utilizing a linear polarization test method. Linear polarization is a nondestructive potentiodynamic electrochemical test method which scans the potential from E_{corr} - 10 to E_{corr} +10. The current is measured on a linear scale and R_p is determined by dividing ΔE by ΔI. A typical linear polarization plot is show in Figure 3.

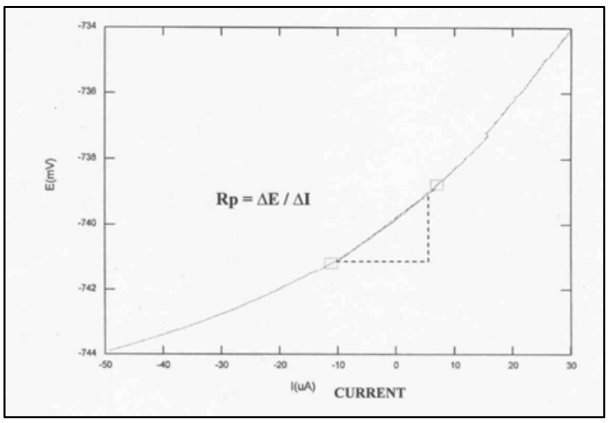
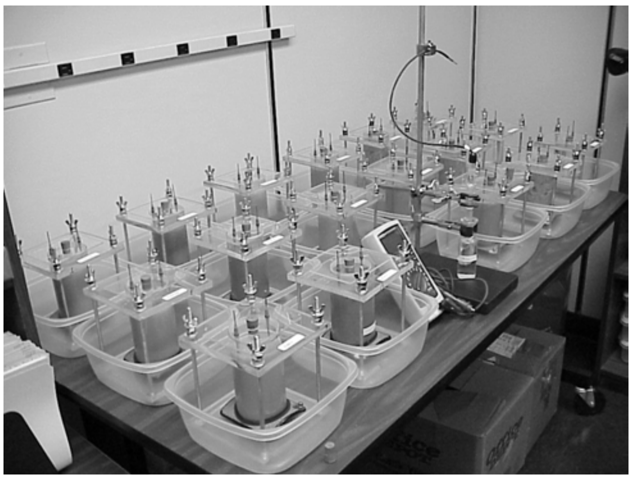


Figure 3 – Typical linear polarization plot showing determination of Rp.

Electrochemical impedance spectroscopy. The effect of drilling fluids under study on the deterioration of standard asphaltic shopcoat and the corrosion rate of ductile iron pipe were evaluated utilizing visual inspection and EIS (Electrochemical Impedance Spectroscopy) electrochemical procedures outlined in ASTM 106 and Princeton Applied Research, EG&G Application Notes (PAR, 2005). The equipment utilized was the same as that previously described with the software being EIS "*Powersine*" supplied by Princeton Applied Research, Oak Ridge, TN (EG&G, 1998).

EIS analysis commonly utilizes three types of graphical plots for analysis of the corrosion cell. These are the Bode Impedance Plot (a plot of the log of impedance versus the log of frequency), the Bode Phase Angle Plot (a plot of the phase angle versus the log of frequency), and the Nyquist Plot (the real component of impedance of the cell versus the imaginary component of impedance of the cell). R_p values from the Nyquist Plot can be utilized to calculate MPY corrosion rates based on equations previously described. Changes in impedance and capacitance over time can be utilized to evaluate the deterioration (or lack thereof) of the coating. The impedance and capacitance values are obtained from the Bode plots.

Electrochemical cell design and construction. Twenty-four specially designed electrochemical cells which could accommodate the curvature of an 8" DIP pipe external surface and an area of approximately 100 cm^2 were constructed for the study. The specially designed and constructed test cells are shown in Figure 4.



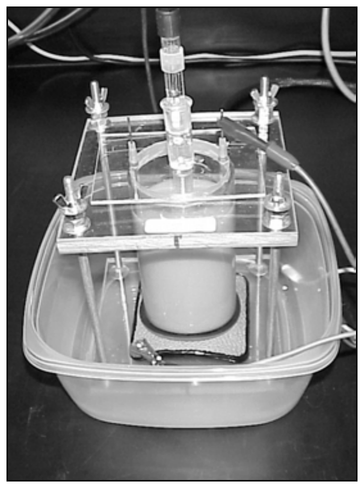


Figure 4 – Photograph of specially designed and constructed electrochemical corrosion cells utilized for HDD fluids study. A typical cell consisted of a DIP coupon, the drill fluid, a platinum counter electrode, and a SCE (saturated calomel reference electrode).

Data Analysis. Weight loss results over the twelve month exposure period were compared by type of exposure (e.g., drilling fluid BB vs. CZ vs. CV, etc.) and also by exposure period (e.g., corrosion rate of fluid BB at one month vs. corrosion rates at 3,6,9, and 12 months). Average weight loss results over the twelve month exposure period were also compared to five different electrochemical test methods over a 1 week, 1 month, 2 months, 3 months, 4 months, and 6 months exposure period to determine if meaningful results could be determined by an electrochemical test method(s) in a shorter time period than the one year test required for weight loss comparisons.

Data analysis was conducted by a combination of visual analysis of graphs and statistical analysis of results. Statistical analysis of the data was conducted utilizing MINITAB, version 14, statistical analysis software manufactured by Minitab, Inc., State College, PA. The weight loss results over the twelve month exposure period were first analyzed to determine if the data could be classified as "normally distributed". Means, medians, variances, and standard deviations were determined. Distribution plots, boxplots, and other statistical plots of the data generated by the software were examined. After determining if the data could be classified as "normally distributed", an ANOVA (Analysis of Variance) examination was conducted to compare the corrosion rates of the five different drilling fluids and the control solution. Both an ANOVA and a "t-test" analysis were utilized to compare the results of the electrochemical corrosion test results with the weight loss results.

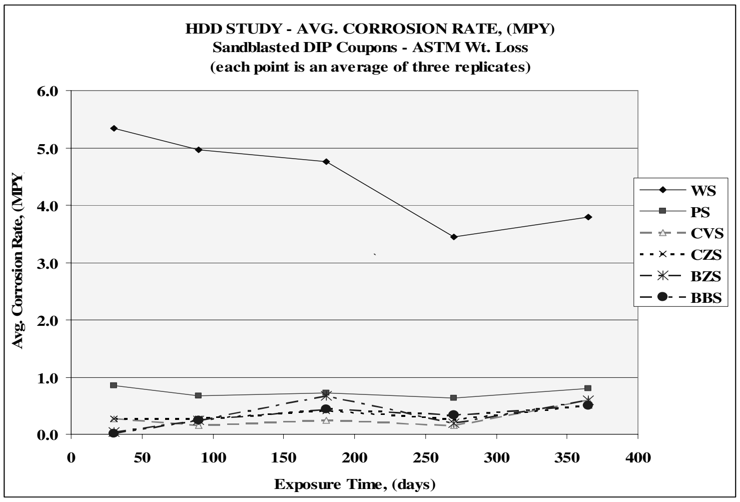


Figure 5 - Average mil per year (MPY) corrosion rate of sandblasted coupons over twelve month exposure. (Based on weight loss).

RESULTS AND DISCUSSION

Weight Loss Analysis and Visual Examination. Weight loss and corrosion rate results of the ASTM G162 weight loss tests are graphically displayed in Figures 5-8 and tabulated in Tables 1-4.

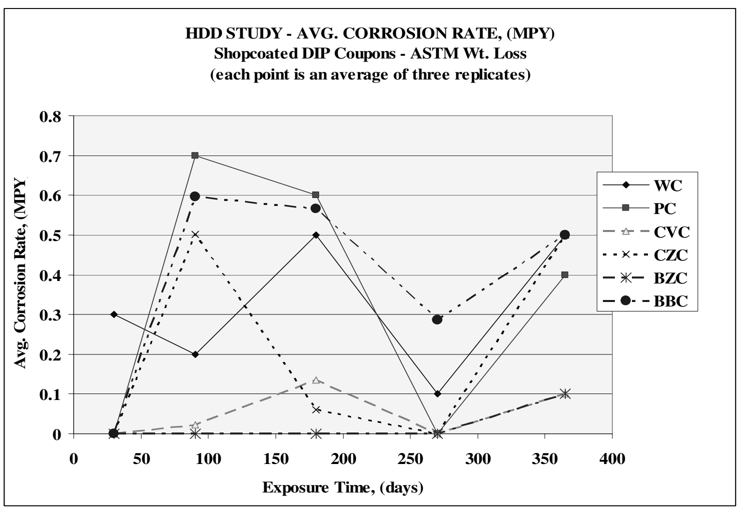


Figure 6 - Average mil per year (MPY) corrosion rate of coated coupons over twelve month exposure. (Based on weight loss).

	TIME OF EXPOSURE (DAYS)				
Environment & Treatment	**30**	**90**	**180**	**270**	**365**
WS	5.3 (0.13)	5.0 (0.13)	4.8 (0.12)	3.4 (0.09)	3.8 (0.10)
PS	0.8 (0.02)	0.7 (0.02)	0.7 (0.02)	0.6 (0.02)	0.8 (0.02)
CVS	0.3 (0.01)	0.2 (0.01)	0.2 (0.01)	0.2 (0.01)	0.6 (0.02)
CZS	0.3 (0.01)	0.3 (0.01)	0.4 (0.01)	0.3 (0.01)	0.5 (0.01)
BZS	0.0 (0.0)	0.2 (0.01)	0.7 (0.02)	0.2 (0.01)	0.6 (0.02)
BBS	0.0 (0.0)	0.2 (0.01)	0.4 (0.01)	0.3 (0.01)	0.5 (0.01)

Table 1. Summary of Average Corrosion Rate in MPY (mm per year) for Blasted Coupons versus Time. (Average of Three Coupons per Exposure)

	TIME OF EXPOSURE (DAYS)				
Environment & Treatment	**30**	**90**	**180**	**270**	**365**
WC	0.3 (0.01)	0.2 (0.01)	0.5 (0.01)	0.1 (0.0)	0.5 (0.01)
PC	0.0 (0.0)	0.7 (0.02)	0.6 (0.02)	0.0 (0.0)	0.4 (0.01)
CVC	0.0 (0.0)	0.0 (0.0)	0.1 (0.0)	0.0 (0.0)	0.1 (0.0)
CZC	0.0 (0.0)	0.5 (0.01)	0.1 (0.0)	0.0 (0.0)	0.5 (0.01)
BZC	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
BBC	0.0 (0.0)	0.6 (0.02)	0.6 (0.02)	0.3 (0.01)	0.5 (0.01)

Table 2. Summary of Average Corrosion Rate in MPY (mm per year) for Coated Coupons versus Time. (Average of Three Coupons per Exposure)

	Blasted Coupons Weight Loss (MPY)			
	Range	Mean	Median	Std. Deviation
WS Coupons	3.3 – 5.5	4.5	4.6	0.8
PS Coupons	0.5 – 0.9	0.7	0.7	0.1
BBS Coupons	0.0 – 0.6	0.3	0.4	0.2
BZS Coupons	0.0 – 0.8	0.4	0.2	0.3
CVS Coupons	0.0 – 0.8	0.3	0.3	0.2
CZS Coupons	0.1 – 0.6	0.4	0.4	0.2

Table 3. – Statistical Summary of Blasted Coupons Weight Loss MPY Data

	Coated Coupons Weight Loss (MPY)			
	Range	Mean	Median	Std. Deviation
WC Coupons	0.0 – 0.7	0.3	0.4	0.2
PC Coupons	0.0 – 0.8	0.4	0.4	0.3
BBC Coupons	0.0 – 0.9	0.4	0.4	0.3
BZC Coupons	0.0 – 0.1	0	0	0
CVS Coupons	0.0 – 0.3	0.1	0	0.1
CZS Coupons	0.0 – 0.6	0.2	0	0.2

Table 4. Statistical Summary of Coated Coupons Weight Loss MPY Data

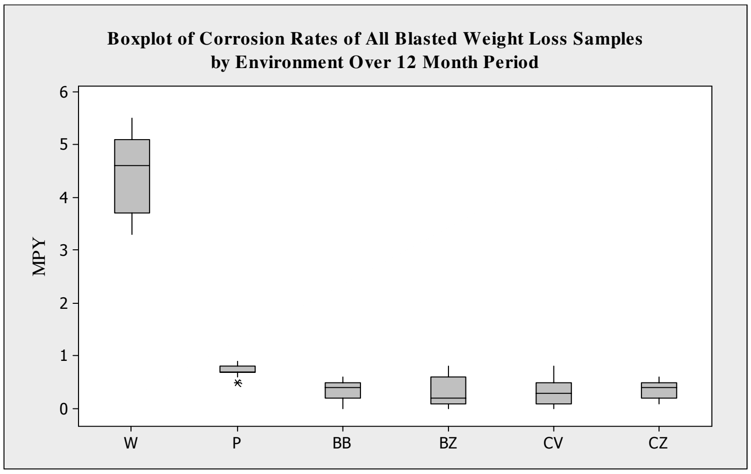


Figure 7 – Boxplot analyusis of corrosion Rates of All Sand-blasted Weight Loss Samples by Environment, Over 12 Month Period.

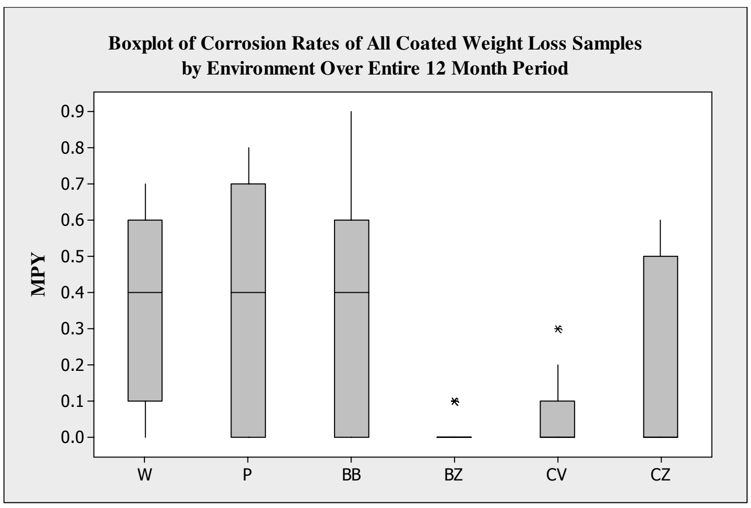


Figure 8 - Boxplot analysis of Corrosion Rates of All Coated Weight Loss Samples by Environment, Over 12 Month Period.

A "Boxplot" representation of data is a quick, visual statistical analysis method to compare sets of data. Boxplot presentations of the MPY data from the ASTM weight loss evaluation are given in Figures 7 and 8. In these figures, the box represents the middle 50% of the data, the horizontal line is the median, the whiskers designate the range, and the asterisks represent the outliers. Outliers are defined as being outside ±1.5 times the middle 50% of the data.

Visual analysis indicated fluids BZ and CV showed no effect on the asphaltic shopcoat over the entire twelve month exposure whereas the shopcoat on coupons in the other fluids were significantly attacked. The shopcoat on all coupons except BZ and CV was approximately 66% gone. The difference in the attack on the asphaltic shopcoat between coupons in the CV fluid versus coupons in the P fluid can be seen in Figure 9.

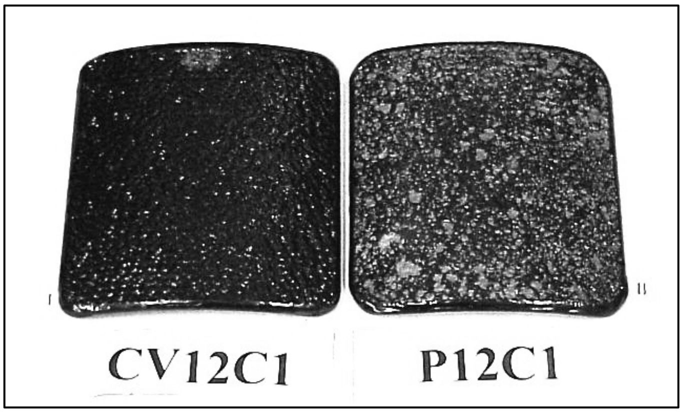


Figure 9 - After 12 months exposure, the asphaltic coating on coupon CV12C1 was unaffected while the asphaltic coating on coupons P12C1 was deteriorated to the point that approximately 66% was missing.

Electrochemical Test Analysis. Two methods of analysis can be utilized for both the polarization resistance test and the EIS test. In the first method, anodic and cathodic Tafel constants of 0.100 are assumed and corrosion rates are determined using the equations given in ASTM G102. Tafel constant values of 0.100 are common for ferrous metals in aqueous environments and are used as default values in the software utilized. In the second method, the Tafel constants obtained from the Tafel scan are utilized in the equations to determine corrosion rate. As a result, a total of five different methods of electrochemical analysis for determining corrosion rate were utilized for the study. These are listed below:

Method 1 – Polarization resistance using assumed Tafel constants of 0.100.
Method 2 - EIS using assumed Tafel constants of 0.100.
Method 3 – Polarization resistance using Tafel constants from Tafel scan.
Method 4 – EIS using Tafel constants from Tafel scan.
Method 5 – Tafel scan using Tafel constants generated by the test.

Preliminary electrochemical testing of the coated coupons revealed this corrosion rate data was extremely inconsistent. The Anderson-Darling Normality Test indicated the coated coupon's weight loss corrosion rate data for four of the six environments were not normally distributed. Therefore, analysis of the electrochemical test results presented here is restricted to the blasted coupons.

To evaluate how corrosion rate results of the blasted coupons varied with time for the various test methods, electrochemical testing was conducted at 1 week, and at 1, 2, 3, 4, and 6 months. For comparison purposes, average corrosion rates calculated from weight loss results for 1, 3, and 6 months were utilized.

Environment & Treatment	1	2	3	4	5
WS	3.6 (0.09)	2.8 (0.07)	6.1 (0.15)	4.9 (0.12)	2.8 (0.07)
PS	1.6 (0.04)	1.0 (0.03)	3.6 (0.09)	2.3 (0.06)	2.0 (0.05)
CVS	0.6 (0.02)	0.4 (0.01)	1.1 (0.03)	0.8 (0.02)	0.6 (0.02)
CZS	3.5 (0.09)	0.6 (0.02)	6.5 (0.16)	1.1 (0.03)	2.3 (0.06)
BZS	14.1 (0.35)	0.9 (0.02)	25.1 (0.63)	1.5 (0.04)	2.7 (0.18)
BBS	1.6 (0.04)	0.8 (0.02)	2.9 (0.07)	1.4 (0.04)	1.9 (0.05)

Table 5 - Mean Corrosion Rate in MPY from Electrochemical Tests

The mean corrosion rate (MPY & mm per year) results for each environment in this study utilizing the five different electrochemical test methods are tabulated in Table 5 above.

SUMMARY AND CONCLUSIONS

Literature Search.

- Trenchless technology methods of pipeline installation, and specifically, horizontal directional drilling methods, are being utilized more and more frequently in the United States. Ductile iron pipe is being utilized more and more for HDD installations due to its' successful service record, impermeability, inherent strength, and toughness.

- Based on this investigation, no previous extensive studies have been conducted on the corrosiveness of HDD drilling fluids to ductile iron pipe.

- Soils corrosive to ductile iron pipe can be identified utilizing procedures outlined in APPENDIX A of AWWA C105, and when conditions warrant, the pipe can be protected utilizing a successful corrosion protection method which is recommended by the iron pipe industry (e.g. polyethylene encasement).

- Various electrochemical test methods are available to determine instantaneous corrosion rates of metals in aqueous environments.

Summary of Weight Loss Test Results.

- None of the five most common drilling fluid "pure" mixtures (i.e. not mixed with soils) evaluated in this study would be considered corrosive to DIP. Average weight loss corrosion rates in all mixtures were less than 1 mil (0.001") per year [0.025 mm/yr.] over the one year test period.

- The standard shopcoat on DIP did provide some extra corrosion protection lowering the weight loss corrosion rate in a 3.5% standard salt water solution from an average of 3.8 MPY (0.095mm/yr.) to an average of 0.5 MPY (0.013mm/yr.).

- The average corrosion rate of sandblasted coupons in the "P" drilling fluid was slightly greater than those in the other four drilling fluids. An ANOVA statistical analysis of blasted coupons indicates the corrosion rates of blasted coupons in fluids BB, BZ, CZ, and CV can be considered statistically the same.

- The weight loss and corrosion rates of sandblasted coupons in the NaCl control solution were approximately four to five times those in the drilling fluids. The variability of the weight losses in the blasted coupons exposed to the NaCl solution were more than those of sandblasted coupons in the drilling fluids.

- While in general the weight loss and corrosion rates were more than the coated samples, weight losses and corrosion rates of the sandblasted coupons contained significantly less variability than the coated coupons between exposure periods. Visual analysis thus proved to be a better method of evaluation for coated coupons than either weight loss or corrosion rate comparisons.

- BZ and CV fluids were less detrimental to the shopcoated coupons than the three other drilling fluids or the control solution. This was confirmed by visual analysis.

- Results indicated the additions of corrosion inhibitors did not change corrosion rates on blasted coupons. However, they did result in significant improvement regarding attack on the asphaltic shopcoat.

Summary of Electrochemical Test Results.

- All five electrochemical test methods consistently produced drilling fluid MPY results higher than those from the corresponding weight loss tests. Therefore, this data indicates drilling fluid MPY results from these electrochemical tests are conservative compared to MPY results from the ASTM weight loss evaluation. This correlation will allow the corrosivity of various drilling fluids on DIP to be evaluated in one or two weeks versus waiting one year or longer for weight loss results.

- The electrochemical MPY results for the NaCl control solution (e.g., “W” coupons) did not follow the above pattern (e.g. electrochemical results being conservative). Depending on exposure time, some electrochemical MPY results in the NaCl solution were higher, and some were lower than corresponding weight loss MPYs.

- Electrochemical test method #2 (e.g., EIS with 0.100 Tafel constants) consistently produced MPY test results nearest to the corresponding weight loss results. In general, polarization resistance methods #1 and #3 produced the least representative MPY results for the drilling fluids. The two EIS methods #2 and #4 produced the results which best represented the weight loss MPY results, and the Tafel analysis method #5 ranked third. However, polarization resistance method #1 did produce representative MPY results with respect to the NaCl control environment.

- With respect to ranking corrosivity of the different environments, methods 2 and 4 were in approximate agreement with the overall 12 month weight loss results which ranked the most corrosive fluids as W and P, and the other four drilling fluids as being approximately equal.

- Pipe-to-soil type potential readings of the coupons in the various environments did not correlate well with corrosion rates and did not prove to be a reliable method to rank the fluids for corrosivity.

Limitations of Results and Suggestions for Further Research. This study was an initial investigation addressing the corrosivity of commonly used HDD drilling fluids for ductile iron pipe installations. Only pure drilling fluid mixtures, mixed in strict accordance with drilling fluid manufacturer's recommendations, were studied. In actual HDD installations, the drilling fluid normally becomes mixed with soil cuttings of the native soil(s), and the characteristics of these native soils may change significantly along the pipeline route. Additional testing is therefore recommended to evaluate corrosivity of combined soil – drilling fluid mixtures which include native soils of varying characteristics. Rather than a lengthy weight loss evaluation, tests utilizing electrochemical test method #2 is recommended to obtain results in a timely manner. Laboratory testing is also recommended to be verified by field evaluation in actual installations.

ACKNOWLEDGEMENTS

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REFERENCES

Allouche, E.N., and Ariaratnam, S.T. (2002). State-of-the-Art-Review of No-Dig Technologies for New Installations. Paper presented at the ASCE Pipelines 2000 Conference (Beneath our Feet: Challenges and Solutions), August 4-7, 2002, Cleveland, Ohio.

ASTM (American Society for Testing and Materials), West Conshohocken, PA.
- ASTM G1–2003 (Standard Practice for Preparing, Cleaning, and Evaluating Corrosion Test Specimens).
- ASTM G-59–2003 (Standard Test Method for Conducting Potentiodynamic Polarization Resistance Measurements).
- ASTM G102–2004 (Standard Practice for Calculation of Corrosion Rates and Related Information from Electrochemical Measurements).
- ASTM G106–2004 (Standard Practice for Verification of Algorithm and Equipment for Electrochemical Impedance Measurements).
- ASTM G162–1999 (Std. Practice for Conducting and Evaluating Laboratory Corrosion Tests in Soils)

AWWA Standards (American Water Works Association, Denver, CO)
- AWWA C105. (1999). "Polyethylene Encasement for Ductile-Iron Pipe Systems"
- AWWA C151. (2002) "Ductile-Iron Pipe, Centrifugally Cast, for Water"

Bonds, R.W., Barnard, L.M, Horton, A.M., and Oliver, G.L. (2004). Corrosion and Corrosion Control Research of Iron Pipe. Paper presented at the ASCE Pipelines 2004 Conference (What's on the Horizon), August 1-4, 2004, San Diego, CA.

Carpenter, R., and Conner, R. (2003). The Versatility of Ductile Iron Pipe in Trenchless Construction. Paper presented at the ASCE Pipelines 2003 Conference (New Pipeline Technologies, Security, and Safety), July 13-16, 2003, Baltimore, MD.

Carpenter, R. (2003). Ductile Iron: Not Just for Open Cut. Editors Log, *Underground Construction*, 58(6) p2.

DIPRA. (2003a). Inspection Report of 45 Year Old 4" Cast Iron Pipe Encased in Loose Polyethylene - Lafourche Parish, Louisiana, May 28, 2003. Ductile Iron Pipe Research Association, Birmingham, AL

EG&G. (1999a). Electrochemical Techniques for Corrosion Measurement. Customer Training Seminar, EG&G Instruments, Inc., Oak Ridge, TN, (May, 1999).

EG&G. (1998). *PowerSine*™ Software. EG&G Instruments, Inc., Oak Ridge, TN.

Gray and Ductile Iron Founders' Society, Inc. (1971). Cleveland, Ohio: Gray and Ductile Iron Castings Handbook.

Griffin, J. (2003). Ductile Iron Proves Effective with HDD, Pipebursting. *Underground Construction*, 58(6), 12-15.

Horton, A.M. (1988). Protecting Pipe with Polyethylene Encasement, 1951-1988. *AWWA Water World News*, *4*(3), 26-28.

Knight, M., Duyvestyn, G., and Gelinas, M. (2001). Excavation of Surface Installed Pipeline. *Journal of Infrastructure Systems*, 9,123-129.

Lawson, G. (2003). Water and Sewer Construction with Horizontal Drilling Equipment. Paper presented at ASCE Pipelines 2003 Conference (New Pipeline Technologies, Security, and Safety), July 13-16, 2003, Baltimore, MD.

PAR. (2005). Princeton Applied Research Application Notes, Oak Ridge, TN
PAR CORR 1, CORR-4, AC-1, AND AC-5 Application Notes

Rowell, R. (2003). Using Ductile Iron Pipe for Horizontal Directional Drilling Installations. Paper presented at the ASCE Pipelines 2003 Conference (New Pipeline Technologies, Security, and Safety), July 13-16, 2003, Baltimore, MD.

Tenbusch, A. and Carpenter, R. (1998). Municipal Infrastructure – Innovative Trenchless Replacement Method Utilizing Bell-Less Ductile Iron Pipe, Case Studies. PLD-ASCE Pipelines in the Constructed Environment, August 1998, San Diego, CA.

U. S. Pipe and Foundry Company. (2004). Guidelines for Use and Application – Horizontal Directional Drilling (HDD) with TR FLEX® Pipe, 4" – 24". Retrieved November 16, 2004, U. S. Pipe website: http://www.uspipe.com.

Cathodic Protection of an Existing Ductile Iron Water Main Using Linear Distributed Anodes

Jeff Schramuk[1] and Mark Johnson[2], P.E.

Abstract

Applying cathodic protection can extend the life of an existing water main, but installing a cathodic protection system in an urban environment can pose significant construction challenges. The City of LaCrosse, WI Water Utility has implemented a demonstration project using horizontal directional drilling equipment to install a continuous linear distributed anode directly alongside an existing water main. The water main is approximately 20 years old, but has experienced significant external pitting corrosion. The results of the initial demonstration project suggest that a 40-year life extension of the water main may be achievable at a cost that is significantly less than the replacement cost of the pipe.

Introduction

In 2004, the American Water Works Association (AWWA) began to conduct a State of the Industry (SOTI) survey (Runge 2004) to track the critical issues that face the water industry. The SOTI survey has shown that an aging water infrastructure has become the major issue facing the water industry today (Runge 2005, 2006). Unfortunately, water infrastructure repairs and replacement are chronically underfunded in the U.S. The U.S. Environmental Protection Agency estimated that the water industry would need over $275 billion over the next 20 years in capital expenses (USEPA 2003), with nearly $184 billion (66%) of this amount going into transmission and distribution systems.

Clearly, there is a need for water utilities to maintain their current infrastructure as they struggle to close the gap between current spending and the future capital needs. A recent study by the U.S. Federal Highway Administration and NACE International concluded that external corrosion of water mains can be effectively mitigated by the application of coatings and cathodic protection (FHWA 2001). This paper demonstrates a unique installation technique where cathodic protection is applied to a corroded 20-year old ductile iron water main in an urban environment at a cost that is much less than replacing the pipe.

Background

In 2003, the LaCrosse Water Utility (LWU) uncovered a water transmission main in order to relocate a portion of the pipe as part of a pending road improvement project. The entire water main runs along a major county trunk highway for approximately

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1-¼ miles (2 km) and consists of both 12-inch (30.48-cm) and 16-inch (40.64-cm) ductile iron pipe (AWWA Class 50) that was installed in 1986. During the excavation, a 5-foot (1.52 m) section of the main was found to have significant wall pitting attributed to external corrosion (**Figure 1**).

In 2005, a metallurgical analysis was performed on this same 5-foot (1.52 m) section of 16-inch (40.64-cm) diameter pipe that had been cut from the water main. The analysis consisted of a visual inspection, chemical analysis of the pipe, tensile and hardness tests, scanning electron microscopy, and a longitudinal metallographic cross section of the pipe wall. The metallurgical analysis concluded that the pipe's wall thickness was reduced to 0.10 inches (0.3 cm) at the thinnest point due to severe localized corrosion with corrosion pit depths ranging to 0.27 inches. This reduction in the wall thickness is equivalent to a 71% wall loss based on an original nominal wall thickness of 0.34 inches (0.9 cm). The outer diameter surface was found to contain elements that are characteristic of dirt or clay in addition to oxidation products.

Most of the pipe is not believed to have been installed with any means of corrosion protection other than the standard 1-mil asphalt coating and the annealing oxide included in the manufacture of this product. The LWU reports that the eastern end of the water main was installed with loose polyethylene encasement (AWWA 1996). The pipe was cable-bonded at each pipe joint to achieve electrical continuity for thawing purposes during extremely cold weather. Electrical continuity of the water main was verified in 2003.

Since a large portion of this water main is located beneath the reconstructed highway, excavating to replace the main would be both disruptive and expensive. Waiting for a failure to occur is not an option since the water main is a primary supply line to the area which it serves. Repairs to the main would also be very costly and could be expected to disrupt traffic and water service along the highway.

Cathodic protection retrofit installations can have sacrificial anodes installed on a reactive basis for pipe replacements after a water main break (AwwaRF 1995). Sacrificial anodes are also being installed proactively at regular intervals to control corrosion on existing water distribution mains (Klopfer 2005). Alternatively, impressed current cathodic protection anodes which are powered by a rectifier-transformer are often installed at a location "remote" from a pipeline that they are intended to protect. These anodes can also be distributed at regular intervals alongside the pipeline. Due to right-of-way constraints, each of these options was not possible. As an alternative, the LWU implemented a demonstration project using horizontal directional drilling (HDD) equipment to install a continuous linear distributed anode (CLDA) system directly alongside the water main. Approximately 1500 feet (457.2 m) of contiguous pipe was considered for this project to assess the effectiveness of such an approach for the remainder of the water main in future years.

Cathodic Protection Design

The CLDA system (**Figure 2**) is factory assembled and consists of 5 basic components. A central #8 AWG-ECTFE insulated copper cable serves as a low-resistance conductor to deliver the required DC current without incurring a substantial longitudinal voltage drop. A continuous copper-cored mixed-metal oxide (MMO) anode wire provides low attenuation for reduced splicing to the #8 copper header cable. Prepackaged calcined petroleum coke breeze surrounds the MMO anode, which enhances the anode's efficiency by carrying the DC current electrolytically from the anode wire. A porous, acid-resistant fabric jacket ("sock") centralizes the anode wire within the coke breeze. Finally, a protective monofilament braiding surrounds the fabric jacket and tightens firmly in place around the coke breeze when pulled by HDD equipment (**Figure 3**).

The CP system has two individual anode branches – one branch for the 300-foot (91.44-m) pipe section (west of the rectifier) and the other for the approximately 1200-foot (365.76-m) pipe section (east of the rectifier). Each anode branch has a second parallel (redundant) #8 AWG-HMWPE cable that is looped and field spliced to the primary anode cable to provide another current path to the MMO anode wire. An insulated #8 AWG-HMWPE stranded copper cable is run between the rectifier's DC negative output terminal and a fire hydrant that is located next to the rectifier.

A conventional air-cooled rectifier energizes the CLDA system. The rectifier and a separate anode junction terminal box are installed near a power pole that provides AC service to the rectifier (**Figure 4**). A common shunt is located on the rectifier's panel board to allow DC current output to be measured using an external multimeter. Separate DC volt-amp meters are installed in the cabinet to display the rectifier's DC outputs.

The two anode branches are connected in parallel via a junction terminal box equipped with power resistors and calibrated shunts that are mounted to an insulated terminal board within a vented NEMA 3R enclosure (**Figure 5**). The power resistors are used to regulate DC current flow between the two different-length pipe sections. The calibrated resistor shunts are used to measure current flow through each anode branch without interrupting either anode circuit. An insulated #8 AWG-HMWPE stranded copper cable connects the bus bar within the junction box to the main DC positive output lug of the rectifier.

Results

The selection of a CP criterion to significantly reduce corrosion rates for a bare or poly-wrapped ductile iron water main is not as exact and typically not as critical as for coated steel pipelines that convey hazardous gases or liquids. For the latter structures, the current-applied (ON) -0.85 volt criterion, the -0.85 volt Instant-Off (I-O) criterion, or the 100 mV polarization decay criterion are the minimum accepted

CP standards (NACE 2002) for government-regulated energy pipelines. However, these NACE criteria are very conservative for the effective control of corrosion for non-regulated structures such as water mains. These structures can tolerate some corrosion deterioration provided there are no service disruptions. For example, CP data recorded over the last 15-20 years in Canada (Raymond 1998 and Wright 1991) has shown that approximately 100 millivolts of current-applied potential shift from the OFF (baseline readings prior to applying CP) readings decreases water main breaks by up to 90-95% during the life of the CP system.

Interpretation of the CP Potential Data

Using a wire reel, OFF pipe-to-soil (P/S) potential data were measured relative to a portable copper-copper sulfate ($Cu\text{-}CuSO_4$) reference electrode placed at 20' intervals along the water main. After allowing the rectifier to operate continuously for approximately two weeks, ON and I-O potential measurements were recorded at these same locations. The effectiveness of the CLDA system (as measured by the degree of polarization of the structure) was also evaluated by calculating the arithmetic difference between the I-O and OFF P/S potentials. These data are shown in graphical form (**Figure 6**). If portions of the main are actually encased in loose polyethylene wrapping, the P/S data represent the potential of the metal surface only at the defects in the poly wrap, but do not necessarily represent the actual potentials at the pipe surface underneath any gaps that are present in the poly wrap.

When considering the polarization profile for the water main, the effectiveness of the CLDA system is effective in allowing 86% of the P/S data to meet or exceed the conservative NACE 100 mV polarization criterion. Areas where the P/S potentials decrease slightly are at points where the water main is valved or is connected to other water mains. This would be expected since these structures add additional metallic surface areas, which decrease the current density applied to the pipe in these areas. Cathodic protection levels are expected to increase over time as the pipe continues to electrochemically polarize.

Individual current outputs were measured at the anode junction box for each of the two anode branches with the rectifier continuously energized. Due to its longer length, the east branch was expected to have an overall lower total resistance to earth (R_T) than the much shorter west anode branch. In actual practice, just the opposite is true. Additional resistance is required for the west leg to "balance" the R_T between the two CLDA branches. A cable-pulling break and the addition of several field splices to the east branch's anode cable are believed to have increased its longitudinal resistance. Field tests, however, verified the anode cables to be electrically continuous after pulling the CLDA into place.

Two mA/ft^2 (21.5 mA/m^2) of pipe surface is often used as a conservative design current density for structures placed in poorly drained soils. However, this applied current density is excessive for in-service polarization levels required to realize an effective service life for ductile iron pipe installed without loose polyethylene

wrapping. Net CP current densities with a properly installed and maintained poly wrapping can be several orders of magnitude less (Schramuk 2005).
If the CP current that is being drained by the interconnected water mains, water service lines, and possible interconnections to the AC power neutral are ignored, the current applied from the entire CLDA system to the subject water main alone can be estimated. Based upon the actual current applied to the water main, the applied current per unit surface area and per unit pipe length can be derived (**Table 1**). It must be noted that, in reality, these interconnections do account for a portion of the total cathodic current. Accordingly, the actual average current per unit surface area and actual average current per unit length will be less than the values shown.

Table 1 - Summary of CLDA Anode Current Applied to the Water Main

Name of Pipe Section	Pipe Diameter	Pipe Length	Sfc. Area of Section	Applied Current	Current by Area	Current by Length
	(in)	(LF)	(ft^2)	(mA)	(mA/ft^2)	(mA/LF)
East	12	1200	3770	5160	1.4	4.30
West	16	300	1257	2310	1.8	7.70

Pipe-to-soil potentials were measured on a natural gas pipeline crossing to determine if the CLDA system was interfering with this foreign pipeline. The gas line's potential shifted negatively less than 20 mV with the CLDA system energized. The local gas utility's corrosion engineer reports no deterioration in the potential data in this area.

Economic Analysis of Applying Cathodic Protection to Existing Pipelines

With proper attention to design, installation and operation, retrofitting an existing pipeline that has high corrosion rates with cathodic protection is a cost-effective and technically-sound alternative to complete main replacement. Cathodic protection is a proactive decision rather than continuing to tolerate and repair main breaks. This approach can significantly curtail and often eliminate the reactive maintenance costs and related social costs (e.g. service disruption and traffic delays during repairs) typically associated with corrosion caused failures in urban areas.

In the case of the LWU demonstration project, a CP system was furnished and installed for the 1500-foot (457.2-m) pipe section at a cost of $47,622 ($31.75/linear foot ($104.17/m)). In comparison, the total cost to replace this section of main is estimated at $192,200 ($128.13/linear foot ($420.38/m)). Thus, the total initial cost of the CP system is less than 25% of the total replacement cost of the main. When expressed over the 40-year life expectancy of the cathodic protection system, the annualized cost of the CP system is only 0.6% per year. These calculations, even though oversimplified, indicate the CP system will extend the life of the water transmission main by at least 40 years at a cost that is much less than replacement of the main. This is particularly noteworthy given that the CLDA cathodic protection

approach can be successfully applied in congested areas with all the technical and economic benefits associated with current day trenchless technologies.
The cost effectiveness of the corrosion control strategy presented here is based on the pipe joints in the test section having effective electrical continuity without the additional costs for excavations to establish the requisite continuity. Unlike sacrificial anode cathodic protection for service life extension, electrical continuity is required to realize corrosion control using the impressed current CLDA system described herein.

Conclusions

The data for the 2006 demonstration project shows that the CLDA system is providing both efficient and cost-effective protection for the water main. The anode manufacturer rates their product for a 40-year operating life. Since the CLDA system is operating well below the manufacturer's recommended DC current output, with proper maintenance, the data suggest that a 40-year life extension of the water main is possible at a cost that is significantly less than the replacement cost of the pipe.

The effective installation of the CLDA anode system relies on the ability to pull the prefabricated coke anode sock, the #8 AWG anode header cable, and the #8 AWG parallel (redundant) cable back through the directionally-drilled hole without over-tensioning the anode cables. A non-conductive, high-tension pulling (mule) tape should be used to pull the anode system back through the "d-bored" hole. Care must be exercised when pulling the CLDA back through the d-bored hole, especially if the anode's coke sock is soaked with either drilling mud or water. This situation could cause the CLDA to be degraded by either damaging the anode's internal factory-spliced connections or any field splices.

Field splices should be minimized in the anode cables to ensure a low longitudinal resistance of the CLDA system, thereby reducing the voltage drop down the length of the anode cable away from the rectifier. The maximum length of any single anode that should be directionally-bored and pulled is about 600 feet (182.88 m), although multiple anode branches can be installed in parallel from a common rectifier.

There is no rigorous cathodic protection monitoring required for this CP system other than verifying that the rectifier remains continuously energized. Reasonable periodic surveillance, possibly including a close-interval potential survey, should be conducted over this section of the transmission main. This action will also confirm that the test stations remain intact and that all wiring connections remain effective.

Based upon the favorable results of the 2006 pilot installation, the LWU anticipates two more phased projects to complete the CP system to address their corrosion concerns for the water main. The design of the next project should commence in early 2007 with bidding and construction to follow. The final project is anticipated for 2008.

References

American Water Works Association Research Foundation (AwwaRF). (1995). "Reducing Main Failures in Brown Deer, Wisconsin," Research Applications #8, Report DS-54/98-8, AwwaRF, Denver, CO.

American Water Works Association (AWWA). (1996). *Manual of Water Supply Practices M41, Ductile Iron Pipe and Fittings*, AWWA, Denver, CO.

Federal Highway Administration (FHWA). (2001). "Corrosion Costs and Preventative Strategies in the United States," Report FHWA-RD-01-156." Office of Infrastructure Research and Development, Washington, D.C.

Klopfer, D. & Schramuk, J. (2005). "A Sacrificial Anode Retrofit Program for Existing Cast Iron Distribution Water Mains." *Journal AWWA*, AWWA, 97:12:50.

NACE International Standard RP0169 (NACE). (2002). "Standard Recommended Practice -Control of External Corrosion on Underground or Submerged Metallic Piping Systems," NACE, Houston, TX.

Raymond, D. (1998). "Cathodic Protection Program, Water Division, Water Distribution Branch Region's Experience," Ottawa-Carleton Utilities Commission: Ottawa-Carleton, Ontario, Canada.

Runge, J. & Mann, J. (2004). "2004 State of the Industry Survey: A Checkup and Guide to Good Health." *Journal AWWA*, AWWA, 96:10:40.

Runge, J. & Mann, J. (2005). "2005 State of the Industry Report: A to Good Health." *Journal AWWA*, AWWA, 97:10:58.

Runge, J. & Mann, J. (2006). "2006 State of the Industry Survey: An Eye to the Future." *Journal AWWA*, AWWA, 98:10:64.

Schramuk, J. & Rash, V. (2005). "Cathodic Protection for a New Ductile Iron Water Transmission Main." *Materials Performance*, NACE, 44:10:20.

U.S. Environmental Protection Agency (USEPA). (2003). "Drinking Water Infrastructure Needs Survey and Assessment," USEPA 816-R-05-001. Office of Ground Water & Drinking Water, USEPA, Washington, D.C.

Wright, A. and Nicholson, J. (1991). "A Report on the Efficiency of the Cathodic Protection System Installed on the Water Mains in Peterborough." Peterborough, Ontario Utilities Commission: Peterborough, Ontario, Canada.

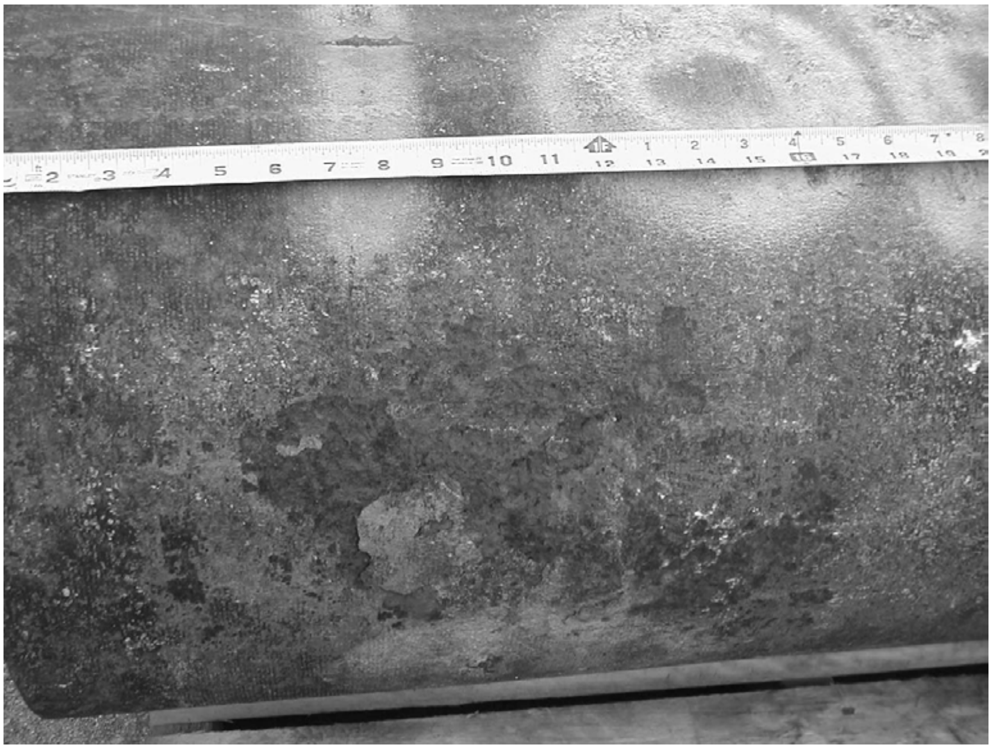

Figure 1 - Corrosive Attack to 16" Ductile Iron Pipe (71% wall max pit depths)

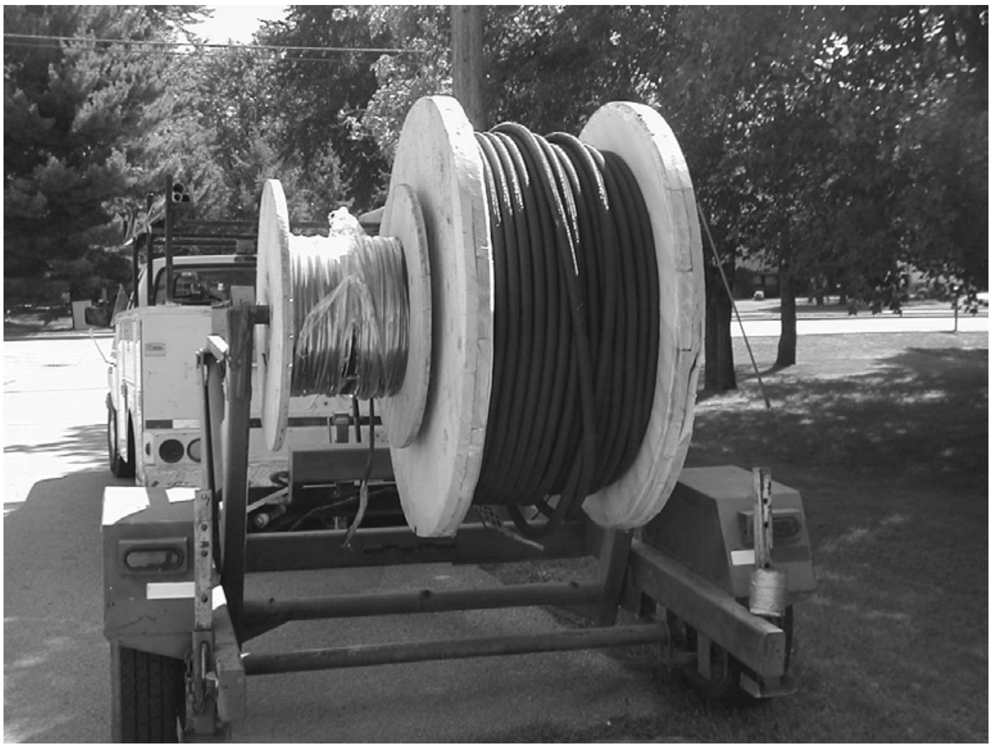

Figure 2 – CLDA Materials (Sock anode on right, parallel cable on left reel)

Figure 3 – Horizontal Directional Boring Equipment

Figure 4 – Rectifier and Anode Box (rectifier negative is connected to hydrant)

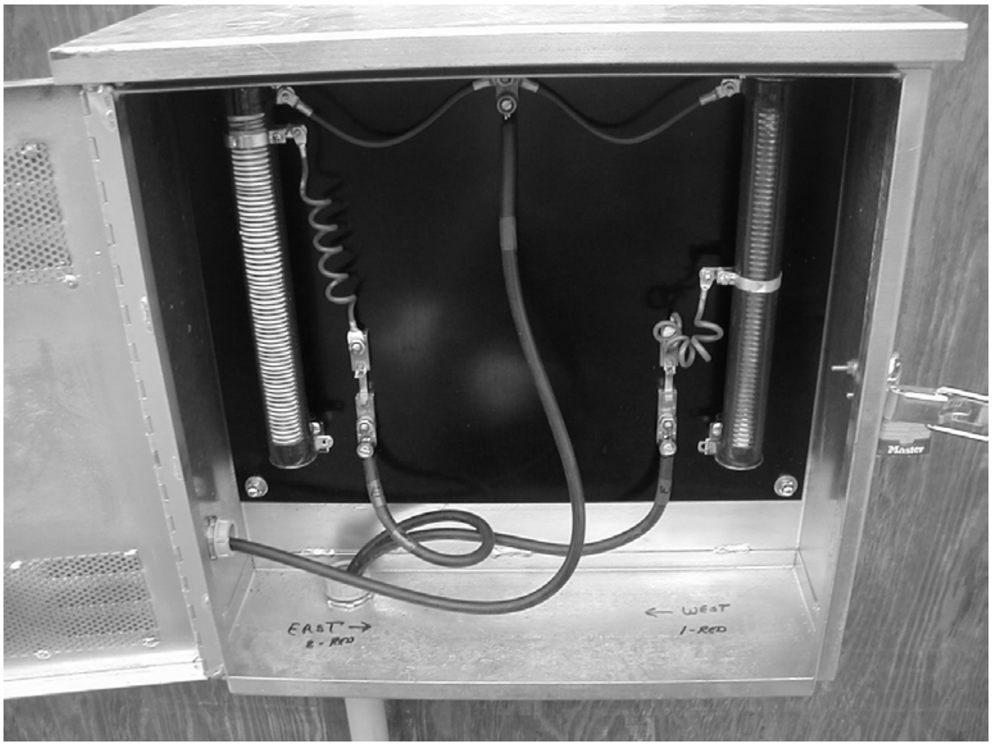

Figure 5 – Anode Junction Terminal Box

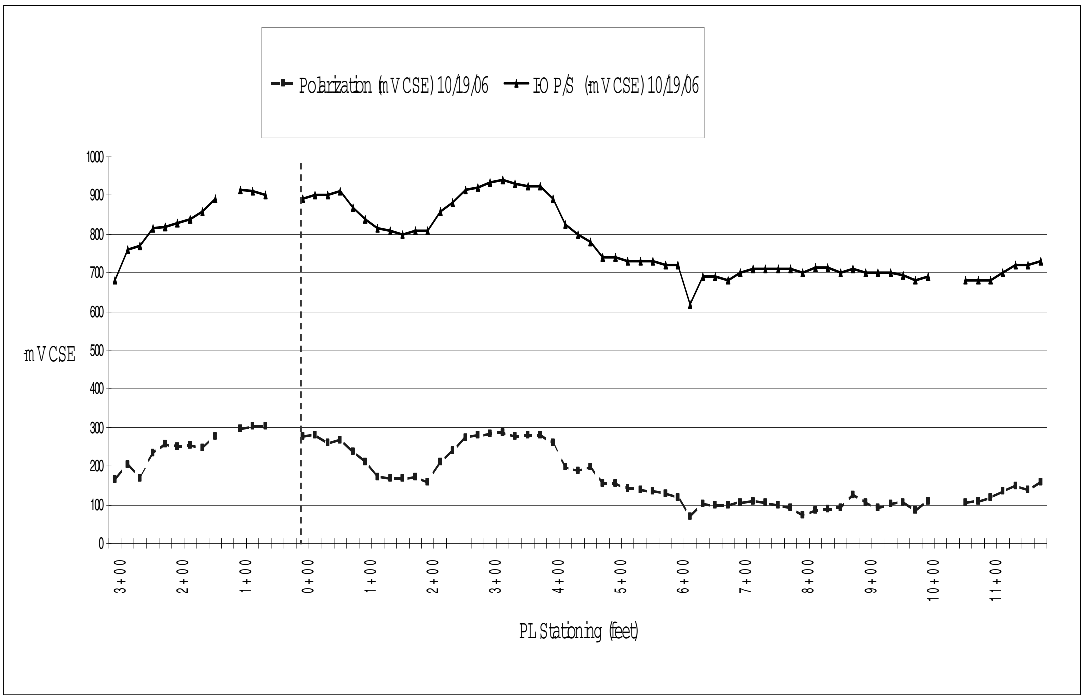

Figure 6 – CP Potential Profile (dashed vertical line is at rectifier STA 0+00)

Corrosion Protection of Large Diameter Welded Steel Pipelines With Cement Mortar Coatings

By: Henry Bardakjian, P.E [1], Michael McReynolds, S.E, P.E [2], and Del Hausmann, P.E. [3]

Abstract

Cement mortar coatings have been widely used in the Western United States since 1935 to protect the exterior of large diameter welded steel pipelines (WSP). Applications include pipe up to 144-inches in diameter and a wide range of environmental conditions.

The Metropolitan Water District of Southern California has had extensive experience with mortar-coated WSP and at present has over 150 miles of this type of pipe in its water transmission system. Characteristics of a recently installed 7-mile, 120-inch- diameter, mortar-coated WSP pipeline are described.

Steel coated with cement mortar is normally protected against corrosion by a passivating iron oxide film that forms and is maintained in a highly alkaline environment of hydrated Portland cement. The potential of passive steel in concrete is several hundred millivolts more positive than the potential of bare or organically coated steel. This unique property makes it possible to identify areas of corrosion activity on mortar-coated WSP and areas where external currents are collecting or discharging.

The long-term performance of cement-mortar coatings is dependent on pipe design, manufacture, and installation. The history of cement mortar coatings is discussed, and recommendations are made for application of mortar-coated WSP.

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Introduction

One of the world's first mortar-lined-and-coated steel pipelines was installed in the City of St. John, New Brunswick, Canada, in 1855 (Figure 1). The 16-ga steel cylinder on this pipeline was riveted and lined with cement mortar, and the exterior was protected with cast-in-place concrete. By the 1920s mortar coatings were applied by the pneumatic spray process ("gunite" or "shotcrete") (Figure 2). In the early 1940's impacted mortar coatings were introduced whereby cement mortar is delivered at high velocity against the rotating steel cylinder surface through two counter rotating wheels or by means of a moving belt. The counter-rotating wheels first consisted of wire brushes, but serrated rubber disks are now in general use (Figure 3). Initially the impacted mortar processes were limited to a maximum pipe diameter of 60 inch, and in the early 1950s they were extended to 144-inch diameter.

Early major users of large diameter mortar coated WSP included the U.S Bureau of Reclamation (USBR), the Metropolitan Water District of Southern California (MWD), East Bay Municipal Utility District, Los Angeles Department of Water and Power, San Diego County Water Authority, City of San Diego, San Francisco Water Department, Contra Costa County Water District, and Southern Nevada Water Authority. Almost every City in the West Coast now has mortar coated WSP in its water system.

Figure 1. 12-inch diameter riveted steel with concrete lining & field applied conc. coating installed in 1855.

Figure 2. Pneumatic mortar application.

Figure 3. Cement-mortar Coating impaction with two counter rotating rubber brushes.

Most agencies had their own specifications for the mortar coating. For instance, the USBR included its mortar coating specification in its Concrete Manual. The American Water Works (AWWA) Standard C205 for mortar lining and coating was issued in 1941 and has now been adopted all or in part by most agencies.

Mortar Coating Components and Reinforcement

The USBR specified mortar mix proportions one part cement to not more than four parts of fine aggregate by weight. AWWA C205 specifies one part cement to not more than three parts of fine aggregate by weight. Initially the minimum moisture content of the mortar coating mix was specified to be 6% of the total dry weight of cement and aggregate. Subsequently the minimum moisture was increased to 7% and the maximum absorption was specified as 10%. Further comprehensive investigations have demonstrated that a 1:3 mortar mix is optimum in terms of absorption of the cured mortar, and increasing cement content results in higher absorption (Bardakjian, 1995).

C205 requires reinforcement with either a single wire applied during mortar application or wire mesh applied in a two-pass application process. Most major agencies mandate the use of wire mesh and the two pass mortar application. In either case the reinforcement must lie within the middle and the middle third of the coating thickness. This is difficult to insure with the single wire application.

For WSP larger than 60 inches in diameter, wire mesh is recommended since it provides reinforcement in both longitudinal and transverse directions, and the two-pass application insures the proper location of the reinforcement within the mortar wall.

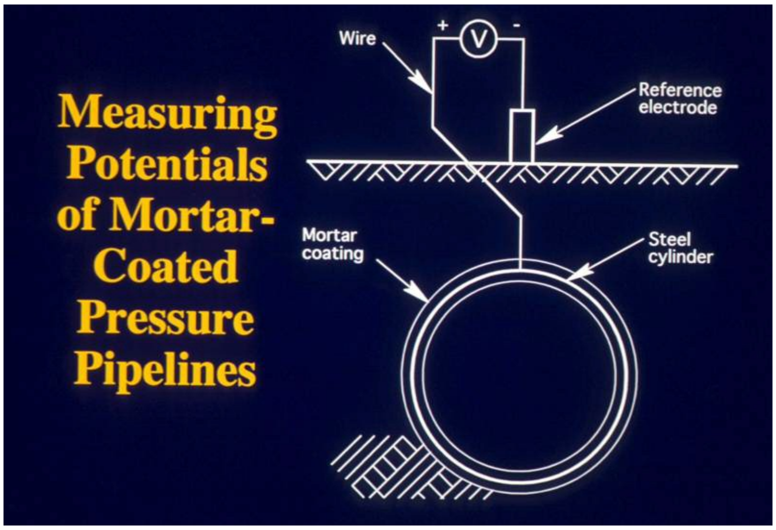

Figure 4. Pipe potential measurements.

Protective Properties of Mortar Coatings

When a section of the City of St. John pipeline was removed after more than a century in service, no significant corrosion was found on interior and exterior surfaces of the steel cylinder. The corrosion-inhibiting properties of concrete and cement mortar were not understood when the pipeline was constructed but are now known to be due to chemical properties of hydrated portland cement. In the highly alkaline cement environment (pH 12.5 to 13.5), steel quickly develops a protective iron oxide film that normally prevents corrosion (Hausmann 1967 &1998). The protective mechanism is so effective that cathodic protection is usually not required on mortar-coated WSP. Where corrosion has occurred it has usually been due to physical damage to the mortar coating or to chlorides penetrating to the steel cylinder in concentrations exceeding a corrosion threshold. Sulfates and other chemicals commonly encountered in the soil do not adversely affect the protective mechanism. Mortar-coated steel is also highly resistant to the collection and discharge of stray currents from such sources as cathodically protected pipelines, rail transit systems, and electrical grounds. The apparent high electrical resistance is due to polarization effects and is largely independent of the ohmic resistance of the mortar coating (Scott 1965). In fact, the effective resistance of mortar-coated steel is typically several hundred times greater than the ohmic resistance of the mortar alone (Hausmann, 1964). Transmission occurs by ion movement in the soil and by electron flow in the steel cylinder, and by convention, current is said to enter the pipeline at cathodic areas and to leave the pipeline at anodic areas. Voltage drops measured along the pipeline can indicate the magnitude and direction of currents carried.

Figure 5. Impacted cement mortar on large diameter WSP.

Significance of Mortar-Coated WSP Potentials

Non-corroding steel in concrete or cement mortar has an electrochemical potential several hundred millivolts more positive (cathodic) than that of bare or organically coated steel. This unique property makes it possible to identify anodic and cathodic areas on mortar-coated WSP by measuring pipeline potentials against a reference electrode as shown in Figure 4. The reference commonly used is a copper sulfate electrode (CSE). For statistical purposes measurements should be made at equal intervals along the pipeline at spacing not greater than the average depth of the pipeline.

In the absence of stray or impressed currents, potentials on mortar-coated WSP in the range of –100 mV to –150 mV (CSE) indicate the steel is passive; potentials more negative than –200 mV imply some steel corrosion activity; and potentials more negative than –350 mV are indication of corrosion. Increasingly negative potentials over time and standard deviations of pipeline potential greater than 10 mV are other indications of active corrosion.

Stray or impressed currents are not damaging to mortar-coated steel until limiting anodic or cathodic polarization potentials are exceeded: Hydrogen embrittlement may occur in high-tensile steel at potentials more negative than –1000 mV (CSE), and steel loss may occur in accordance with Faraday's law at potentials more positive than approximately +600 mV, the polarization potential at which oxygen is generated by consumption of alkalinity in the hydrated cement (Hausmann 1964). Usually when interference is detected cathodic protection is applied to prevent current discharge from the pipeline.

Figure 6. Unloading 120-inch mortar coated WSP for MWD's Pipeline No. 6.

Base potentials along a new pipeline should be established following the first rainy season after installation. Potentials should be measured periodically thereafter, the time interval depending on knowledge of soil resistivity, stray current sources, and prior potential measurements. Under most circumstances there is no justification for applying cathodic protection to mortar-coated WSP until pipeline potential measurements indicate that corrosion threatening integrity of the pipeline is occurring.

Additional Advantages of Cement-Mortar Coatings

In addition to protection against corrosion, mortar coatings increase pipe stiffness and resistance to impact damage. They are also used over dielectric coatings to provide protection against physical damage and soil stresses. This application is also addressed in the AWWA C205 standard.

MWD's Large Diameter Mortar-Coated WSP Pipelines

MWD, one of the largest water agencies in the U.S, has over 800 miles of tunnels, canals, and pipelines in its water transmission system and includes over 150 miles of mortar-coated WSP pipelines 60 inch through 144 inch in diameter.

The most recent large-diameter mortar-coated WSP in the MWD system is the upper reach of pipeline No. 6, consisting of 7 miles of 120-inch pipeline with the following characteristics and features:

- The connection for this raw water line was made at the Skinner filtration plant through a 162-inch x 144-inch tee connection into an existing 162 inch diameter raw water line.
- The system included a 144x144 cross for future connections.

Figure 7. Bedding preparation and installation of 120-inch WSP for pipeline No.6.

- The 120-inch WSP, with a cylinder inside diameter of 121.5 inches, had a standard length of 40 ft.
- The steel cylinder thickness varied between 0.50 inch and 0.75 inch. The 0.50 inch thickness was the minimum thickness required for handling.
- The maximum design circumferential stress for maximum pressure was 21,000 psi.
- The shop-applied mortar coating thickness was one-inch applied in two applications reinforced with 2 in. x 4 in., W1.4xW1.4 welded wire reinforcement (Figure 5)
- The field- applied cement mortar lining thickness was 0.75 inch.
- The field joints were single lap-welded joints except in thrust restraint areas it was double lap-welded.
- Backfill material was select granular material compacted through water jetting (Figure 9).

MWD's mortar-coated WSP pipelines have performed well for over seventy years. However at locations, where stray current interference has been detected, cathodic protection has been installed to prevent steel loss due to current discharge.

Proper Pipe Design, Manufacture, and Installation of Mortar Coated WSP

Like all types of piping material, the long-term performance of mortar-coated WSP is dependent on proper pipe design, manufacture and installation.

Figure 8. The joint grout bands are filled with cement mortar gout ahead of backfilling with granular material through a Shute.

Pipe Design

- It is recommended that strain or stress due to internal pressure be limited to 700 micro-strains or 21,000 psi. Properly bedded and backfilled pipe with proper mortar reinforcement can tolerate higher strains.
- It is recommended that pipe deflection after bedding and compaction be limited to 2%.
- Minimum cylinder thickness for handling and compaction is dependent on many factors. The cylinder diameter to thickness ratio (D/t) should not exceed 240.
- All steel elements in the pipeline should be electrically continuous to monitor steel potentials and line currents. The joints of larger-diameter steel pipelines are field welded which automatically achieves the required continuity.
- Mortar-coated steel pipe should be electrically isolated from steel elements not coated with portland cement materials in order to prevent galvanic cells damaging to the steel elements.

Pipe Manufacture

- The steel cylinder should be fabricated with close circumferential tolerances.
- The bare or mortar lined cylinder should be internally-stulled to limit the steel cylinder out-of roundness to a maximum of 1% prior to the application of the mortar coating.
- The mortar coating should be applied in two applications. A 3/8 inch thick flash coat should be mechanically impacted first.

- It is recommended that immediately preceding the application of the flash coating, slurry composed of 94 lb of Portland cement to not more than 10 gallons of water be applied uniformly over the steel surfaces. This will start the passivating process of the steel process and also serve as a bonding agent. This requirement is not in the AWWA C205 standard.
- 2 in. x 4 in. welded wire reinforcement should be wrapped over the mortar and tied before the application of the second 3/8-inch or 5/8-inch mortar coating. The size of the welded wire reinforcement is dependent on the size of the pipe. The minimum size of the wire should not be less W 0.9.
- The completed pipe should be lifted and handled with a Forklift fitted with a padded saddle support.
- The pipe is then moved to the curing area.

Installation

- Although mortar coatings can resist physical damage, the pipe should be handled with two nylon slings and a spreader (Figure 6).
- After the foundation and base bedding is prepared, partial bedding is prepared with holes or discontinuity at the joints to allow for welding and installation of the joint grout bands (Figure 7).
- After pipe joints are welded, joint grout bands are installed, and the joints are grouted, the backfill and compaction in the pipe zone can begin.
- Backfilling and compaction in the pipe zone can be performed by different procedures. Figure 8 shows sluicing the granular material in the haunch areas and then water jetting for consolidation. Backfilling in lifts and compaction through mechanical compactors can be also be used. The backfill and compaction in the pipe haunch area is very important.
- After backfilling is complete and pipe deflection is verified, the internal stulling can be removed.

Conclusions and Recommendations

- Cement mortar coatings have been successfully used to prevent the corrosion of large-diameter WSP for over seventy years.
- Cathodic protection is usually not required but may be imposed in very low resistance soils or stray current interference.
- The long term performance of mortar-coated large diameter WSP is dependent on proper pipe design, manufacture, and installation.
- The mortar coating of large diameter WSP should be applied in two applications and should be reinforced with 2 in. x 4 in. welded wire reinforcement. The size of the wire can vary between W.9 and W 1.4 depending on the pipe diameter.
- The uniform application of a slurry composed of 94 lbs of Portland cement to not more than 10 gallons of water over the exterior steel surfaces immediately prior to the application of cement mortar coating is recommended.

Figure 9. Water jetting of the backfill can be observed in the background.

References:

American Water Works Association (2000). AWWA C205-00 Standard, "Cement-mortar Protective Lining and Coating for Steel Water Pipe-4 in. and Larger-Shop Applied," Denver, CO.

Bardakjian, Henry (1995). "Quality Enhancements of Cement-Mortar Coatings", *Proceedings, Second International Conference*, ASCE Pipeline Division Conference, Bellevue, Washington, 734-744.

Scott, Gordon N. (1965). "Corrosion Protective Properties of Portland Cement Concrete", *J. Am. Water Works Assoc.*, 57(8), 1038-1052.

Hausmann, D.A. (1964). "Electrochemical Behavior of Steel in Concrete", *J. Am. Concrete Inst.,* 61(2), 171-188.

Hausmann, D.A. (1967). "Steel Corrosion in Concrete." *Materials Protection*, NACE, 6(11), 19-23.

Hausmann, D.A. (1998). "A Probability Model of Steel Corrosion in Concrete." *Materials Performance,* NACE, 37(10), 64-68.

Bushyeager, M.L. and Risner, W.E. "Distribution Pipeline System Corrosion Monitoring and Mitigation Procedures of a Large Potable Water Agency", *Corrosion 89*, NACE.

ASSESSING POLYETHYLENE ENCASED DUCTILE IRON PIPELINES

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ABSTRACT

This paper presents investigations to ascertain the corrosion control benefits of polyethylene encasement for ductile iron pipelines in corrosive soil environments. Investigative procedures including cell-to-cell potential surveys, side drain technique measurements, in-situ and laboratory soil tests, pipe-to-soil potential measurements and excavation inspections. Included are testing results and the location sites of each investigation. These investigations demonstrate the effectiveness of polyethylene encasement as a corrosion control system for ductile iron pipe.

INTRODUCTION

For more than 50 years ductile iron pipe has been the chosen material for modern water transmission and distribution systems. Advantages such as strength, durability, and reliability make ductile iron pipe the industry standard.

A ferrous material, ductile iron pipe can be subject to corrosion when installed in aggressive environments. When corrosive environments are encountered and identified, the iron pipe industry recommends appropriate corrosion control. The most commonly used form of corrosion control for ductile iron pipe involves the use of polyethylene encasement.[1-4] An American Water Works Association (AWWA) Engineering and Construction Division survey reported 95% of the utilities polled used polyethylene encasement for their corrosion protection of ductile iron pipe.[5]

Many tools and procedures are available to help in the identification of corrosive soil conditions and their subsequent consequences. Parameters such as soil resistivity, pH, moisture content, oxidation-reduction potential, sulfides, chlorides, sulfates, etc., can all be measured and evaluated by the experienced corrosion engineer.[6, 7, 8]

After pipelines are installed, there are also tools to evaluate the condition of the pipe. Water utility break records and maintenance reports can give valuable insight concerning the condition of a pipe system. Survey methods and techniques such as pipe-to-earth potential measurements, close interval surveys, cell-to-cell surveys, side-drain technique, etc., have also been used.[9,10] Ductile iron pipelines generally are not electrically continuous because of their rubber-gasketed, bell-and-spigot installation design. For this reason, test stations generally are not installed for ductile iron pipeline systems. These are important considerations in choosing and evaluating a condition survey procedure for ductile iron pipelines.

This paper documents investigations at three prominent water utilities that practice corrosion control of their ductile iron pipe systems: BexarMet Water District in San Antonio, Texas; Charleston Water System, South Carolina; and Onondaga County Water Authority in Syracuse, New York. The predominant system of corrosion control utilized by these three utilities is polyethylene encasement in accordance with the ANSI/AWWA C105/A21.5 Standard.[6]

PROCEDURES

At each of the investigation sites, there was polyethylene-encased ductile iron pipe, minimal traffic control issues and reasonable excavation access. Utility personnel had determined the location of the pipeline at each site and flagged it accordingly. All other underground utilities in the vicinity of the investigation routes were also located and identified. Discussions with other utilities' personnel and field observations determined that no sources of potential stray current were in the general area.

Since these pipelines, like most ductile iron pipelines, were not installed with joint-continuity bonds or test stations, cell-to-cell potential surveys (Figures 1 – 7) and side-drain techniques (Tables 1, 5 & 9) were performed to search for corrosion. The cell-to-cell survey intervals were five feet directly over the pipeline and with a perpendicular distance of 10 feet on both sides of the pipeline for side-drain measurements. A high impedance voltmeter and two matching copper/copper sulfate half cells were used for the survey. Data from the surveys was directly entered into a notebook computer from which a finished graph was generated for evaluation and locations for excavation (Figures 1 – 7).

In theory, the cell-to-cell potential survey is intended to identify areas of active corrosion that would correspond to locations where the polyethylene encasement was damaged. On the survey graph such areas should be located where the potential shifts from positive to negative (Figures 1 – 7).

After specific site locations were excavated, additional pipe-to-earth potential measurements were conducted (Tables 2, 6 & 10). At all excavations the *in situ* soil resistivity was measured (Tables 3, 7 & 11). Representative soil samples were also obtained and tested for resistivity, pH, and oxidation-reduction potential, along with qualitative tests for sulfides, chlorides, and moisture (Tables 3, 7 & 11).

At each of the excavation sites the soil was carefully removed from around the pipe so that the condition of the polyethylene encasement could be evaluated. A representative sample of the exhumed polyethylene material was tested for thickness, elongation, and tensile strength to determine if it conformed to the requirements of the ANSI/AWWA C105/A21.5 standard, which was in effect at the time of its installation (Tables 4, 8 & 12). The polyethylene material was also evaluated to determine if it exhibited any deterioration. After inspection and removal of the installed polyethylene encasement material, the pipe surface was cleaned and examined. The examination procedures included wire brushing, probing with a pointed hammer, measuring pipe-to-earth potentials, and recording the depth of any corrosion-related pitting. At the conclusion of the inspections, new polyethylene encasement was installed and the pipe inspection site backfilled.

Investigation #1
November 14 – 16, 2005
BexarMet Water District - San Antonio, Texas

For this investigation, the BexarMet Water District made available approximately one mile of right of way containing a polyethylene-encased 16-inch diameter ductile iron water main along the north side of Somerset Road in southern Bexar County. The pipeline was installed in 1986. Three specific sites were identified for excavation.

Results

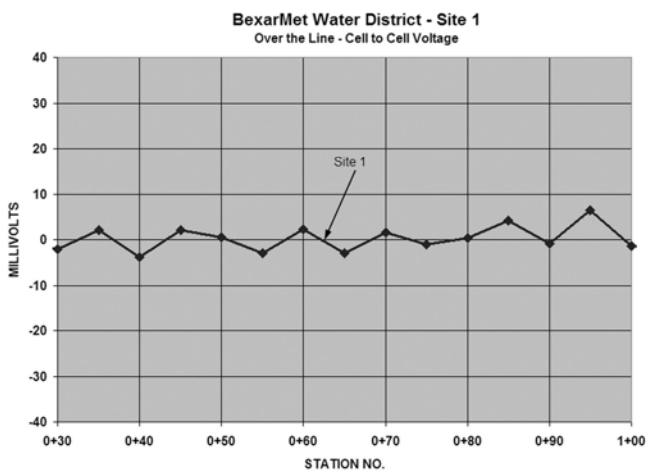

Figure 1 – BexarMet Water District (Site 1)

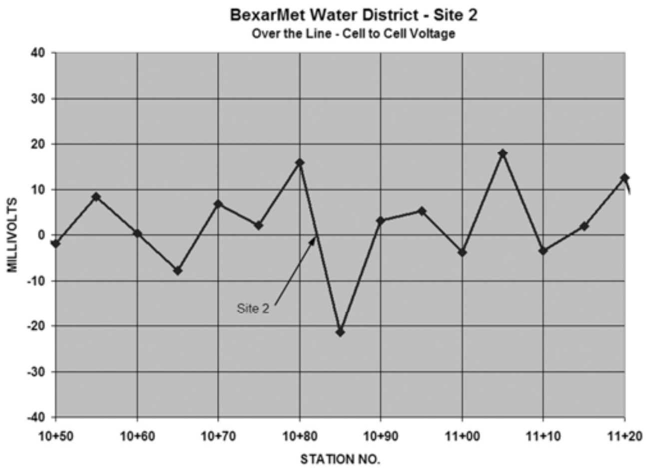

Figure 2 – BexarMet Water District (Site 2)

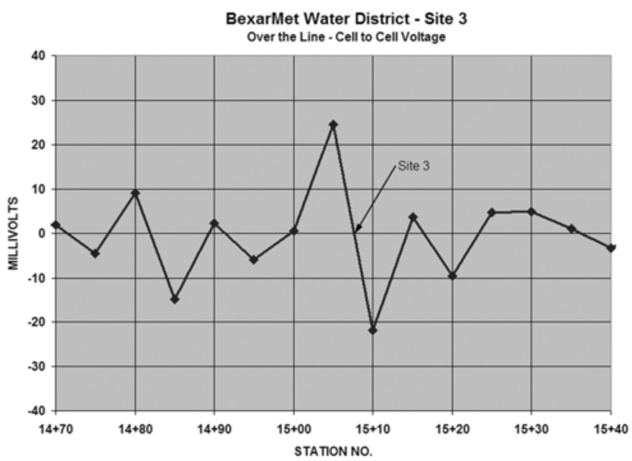

Figure 3 – BexarMet Water District (Site 3)

Table 1 - Side Drain Technique Measurements – BexarMet Water District

Offset Direction	Cell to Cell Potential*		
	Site 1 (Station 0+62)	Site 2 (Station 10+82)	Site 3 (Station 15+08)
10 feet (3.05 m) North	+ 4.5 mV	+ 34.0 mV	- 14.8 mV
10 feet (3.05 m) South	+ 9.7 mV	- 6.2 mV	+ 10.2 mV
*Positive reading indicates anodic area, negative reading indicates cathodic area.			

Table 2 – Pipe-to-soil Potentials – BexarMet Water District

Site	Station	Pipe-to-soil*
1	0+62	- 526 mV
2	10+82	- 518 mV
3	15+08	- 509 mV
*measured with a copper/copper sulfate half cell		

Table 3 - Soil Test Results – BexarMet Water District

	Site 1	Site 2	Site 3
Station Number	0+62	10+82	15+08
4-pin Resistivity [5-ft (1.52 m) spacing]	4,692 ohm-cm	3,064 ohm-cm	4,309 ohm-cm
4-pin Resistivity [10-ft (3.05 m) spacing]	3,734 ohm-cm	2,873 ohm-cm	3,437 ohm-cm
Soil Box Resistivity	1,440 ohm-cm	1,480 ohm-cm	1,400 ohm-cm
pH	7.5	7.6	7.5
Redox	+ 260 mV	+ 160 mV	+ 180 mV
Sulfides	Negative	Negative	Negative
Chlorides	Negative (< 50 ppm)	Negative (< 50 ppm)	Negative (< 50 ppm)
Moisture	Moist	Moist	Moist
Note: All soil samples would be classified as aggressive to ductile iron pipe when using the soil box resistivity values per Appendix A of the ANSI/AWWA C105/A21.5 Standard.			

Table 4 – Polyethylene Encasement Material Tests – BexarMet Water District

SITE 1 (Station 0+62)			
	Transverse	Longitudinal	Standard Minimum*
Thickness, in. (mm)	0.0081 (0.206)	0.0082 (0.208)	0.0072 (0.183)
Tensile, psi (Mpa)	2,097 (14.46)	2,134 (14.71)	1,200 (8.27)
Elongation (percent)	465	445	300
SITE 2 (Station 10+82)			
	Transverse	Longitudinal	Standard Minimum*
Thickness, in. (mm)	0.0083 (0.211)	0.0079 (0.201)	0.0072 (0.183)
Tensile, psi (Mpa)	2,318 (15.83)	2,482 (17.11)	1,200 (8.27)
Elongation (percent)	572	465	300
SITE 3 (Station 15+08)			
	Transverse	Longitudinal	Standard Minimum*
Thickness, in. (mm)	0.0079 (0.201)	0.0076 (0.193)	0.0072 (0.183)
Tensile, psi (Mpa)	2,135 (14.72)	2,534 (17.47)	1,200 (8.27)
Elongation (percent)	650	464	300
*Minimum values per ANSI/AWWA C105/A21.5 Standard at the time of construction.			

No damage was observed to the installed polyethylene encasement at these inspection locations. The soil test results indicated an aggressive environment, but the exposed 16-inch ductile iron pipe was found to be in excellent condition beneath the polyethylene wrap. The pipe surface was moist and clean, with only minor surface oxidation present. Upon sounding and probing of the pipe surface it was revealed that no pitting or graphitization had occurred.

Investigation #2
March 28 – 30, 2006
Charleston Water System - Charleston, South Carolina

The Charleston Water System made available approximately a half-mile section of polyethylene-encased 12-inch ductile iron water main along the south side of Iroquois Street in the Liberty Homes area of Charleston. The pipeline was installed in 1984. Two specific sites were identified for excavation.

Results

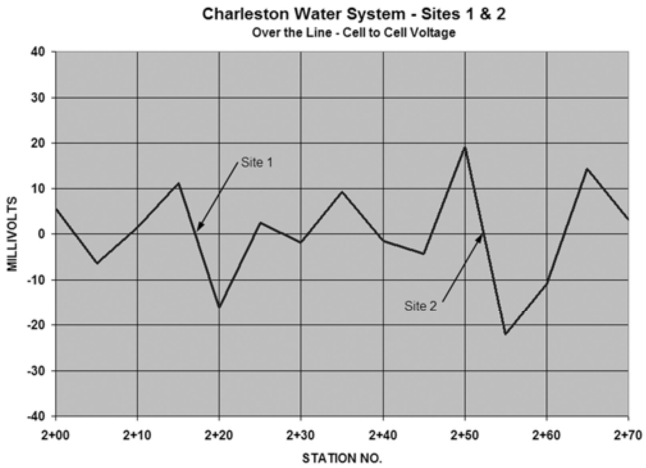

Figure 4 – Charleston Water System (Sites 1 & 2)

Table 5 - Side Drain Technique Measurements – Charleston Water System

Offset Direction	Cell to Cell Potential*	
	Site 1 (Station 2+15)	Site 2 (Station 2+50)
10 feet (3.05 m) North	- 8.6 mV	+ 17 mV
10 feet (3.05 m) South	+ 6.0 mV	+ 7.5 mV
*Positive reading indicates anodic area, negative reading indicates cathodic area.		

Table 6 – Pipe-to-soil Potentials – Charleston Water System

Site	Station	Pipe-to-soil*
1	2+15	- 538 mV
2	2+50	- 544 mV
*measured with a copper/copper sulfate half cell		

Table 7 - Soil Test Results – Charleston Water System

	Site 1	Site 2
Station Number	2+15	2+50
4-pin Resistivity [5-ft (1.52 m) spacing]	4,021 ohm-cm	
4-pin Resistivity [10-ft (3.05 m) spacing]	3,926 ohm-cm	
Soil Box Resistivity	1,480 ohm-cm	1,440 ohm-cm
pH	6.8	5.0
Redox	+ 280 mV	+ 310 mV
Sulfides	Negative	Trace
Chlorides	Negative (< 50 ppm)	Negative (< 50 ppm)
Moisture	Moist	Moist
Note: All soil samples would be classified as aggressive to ductile iron pipe when using the soil box resistivity values per Appendix A of the ANSI/AWWA C105/A21.5 Standard.		

Table 8 - Polyethylene Encasement Material Tests – Charleston Water System

SITE 1 (Station 2+15)			
	Transverse	Longitudinal	Standard Minimum*
Thickness, in. (mm)	0.0075 (0.191)	0.0077 (0.196)	0.0072 (0.183)
Tensile, psi (MPa)	2,478 (17.09)	2,503 (17.26)	1,200 (8.27)
Elongation (percent)	606	552	300
SITE 2 (Station 2+50)			
	Transverse	Longitudinal	Standard Minimum*
Thickness, in. (mm)	0.0076 (0.193)	0.0074 (0.188)	0.0072 (0.183)
Tensile, psi (MPa)	2,371 (16.35)	2,381 (16.42)	1,200 (8.27)
Elongation (percent)	587	493	300
*Minimum values per ANSI/AWWA C105/A21.5 Standard at the time of construction.			

No damage to the installed polyethylene encasement was observed at either of the two locations. The soils were found to be aggressive, but the exposed 12-inch-diameter ductile iron pipe was found to be in excellent condition beneath the polyethylene wrap. The pipe surface displayed only minor surface oxidation. Sounding and probing of the pipe revealed no pitting or graphitization.

At the second excavation site, an abandoned three-inch cast iron pipeline was discovered approximately four feet north of the 12-inch ductile iron pipe. Records indicated that the three-inch pipeline was installed in the 1950s and was, therefore, approximately 55-years old. The three-inch pipe did not have any corrosion protection and had experienced corrosion-related pitting. At a small area cleaned for inspection, the deepest corrosion pit measured 0.18-inches, which would equate to a corrosion rate of 3.3 mils per year.

Investigation #3
May 9 – 11, 2006
Onondaga County Water Authority - Syracuse, New York

The Onondaga County Water Authority made available approximately one mile of right of way containing a polyethylene-encased, 8-inch ductile iron water main along the north side of Hayes Road in Lysander, New York. The pipeline was installed in 1988. Three specific sites were identified for excavation.

<u>Results</u>

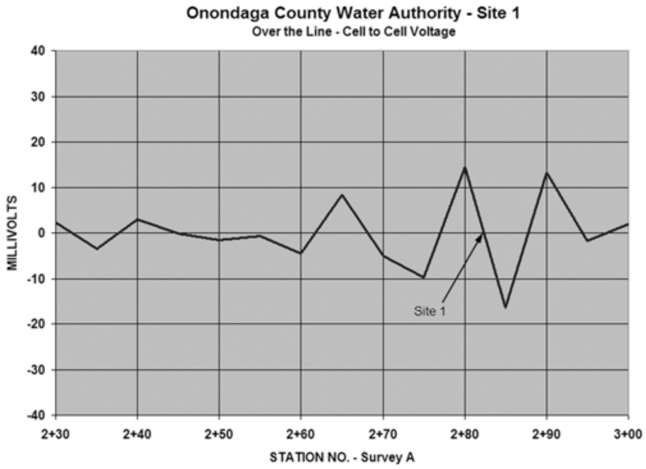

Figure 5 – Onondaga County Water Authority (Site 1)

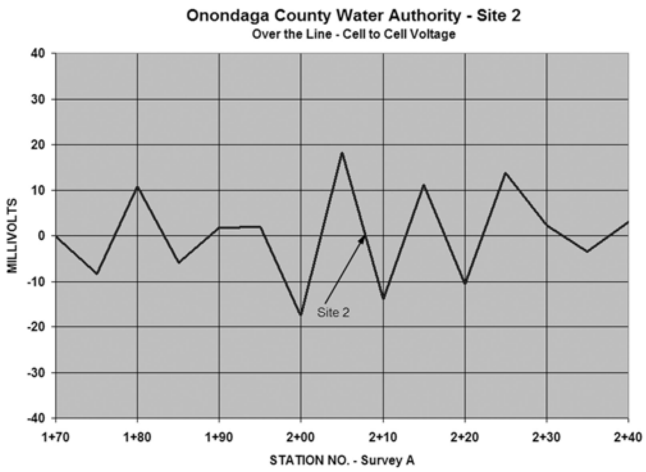

Figure 6 – Onondaga County Water Authority (Site 2)

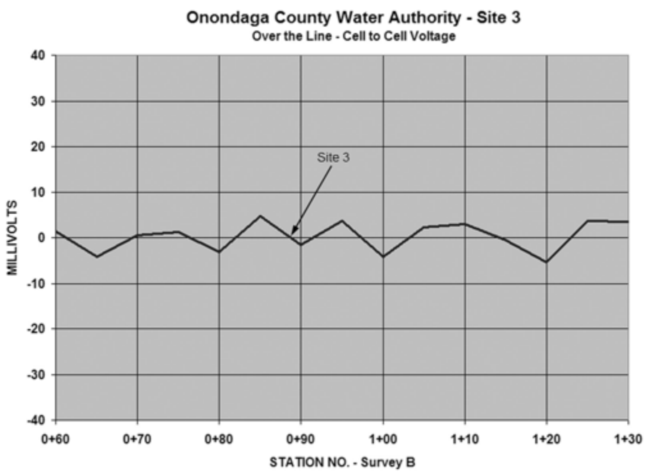

Figure 7 – Onondaga County Water Authority (Site 3)

Table 9 - Side Drain Technique Measurements – Onondaga County Water Authority

Offset Direction	Cell to Cell Potential*		
	Site 1 (Station 2+80A)	Site 2 (Station 2+10A)	Site 3 (Station 0+88B)
10 feet (3.05 m) North	+ 27.2 mV	+ 14.0 mV	- 14.8 mV
10 feet (3.05 m) South	N/A (pavement)	N/A (pavement)	N/A (pavement)
*Positive reading indicates anodic area, negative reading indicates cathodic area.			

Table 10 - Pipe-to-soil Potentials – Onondaga County Water Authority

Site	Station	Pipe-to-soil*
1	2+80A	- 576 mV
2	2+10A	- 564 mV
3	0+88B	- 558 mV
*measured with a copper/copper sulfate half cell		

Table 11 - Soil Test Results – Onondaga County Water Authority

	Site 1	Site 2	Site 3
Station Number	2+80A	2+10A	0+88B
4-pin Resistivity [5-ft (1.52 m) spacing]	8,522 ohm-cm		6,894 ohm-cm
4-pin Resistivity [10-ft (3.05 m spacing]	2,107 ohm-cm		2,873 ohm-cm
Soil Box Resistivity	680 ohm-cm	760 ohm-cm	980 ohm-cm
pH	5.5	6.2	7.1
Redox	+ 190 mV	+ 140 mV	+ 170 mV
Sulfides	Positive	Positive	Negative
Chlorides	Positive (> 100 ppm)	Positive (> 100 ppm)	Trace (50 to 100 ppm)
Moisture	Saturated	Saturated	Saturated
Note: All soil samples would be classified as aggressive to ductile iron pipe when using the soil box resistivity values per Appendix A of the ANSI/AWWA C105/A21.5 Standard.			

Table 12 - Polyethylene Encasement Material Tests – Onondaga County Water Authority

SITE 1 (Station 2+80A)			
	Transverse	Longitudinal	Standard Minimum*
Thickness, in. (mm)	0.0078 (0.198)	0.0076 (0.193)	0.0072 (0.183)
Tensile, psi (MPa)	1,827 (12.6)	2,120 (14.62)	1,200 (8.27)
Elongation (percent)	476	667	300
SITE 2 (Station 2+10A)			
	Transverse	Longitudinal	Standard Minimum*
Thickness, in. (mm)	0.0073 (0.185)	0.0072 (0.183)	0.0072 (0.183)
Tensile, psi (MPa)	2,302 (15.87)	1,739 (11.99)	1,200 (8.27)
Elongation (percent)	640	502	300
SITE 3 (Station 0+88B)			
	Transverse	Longitudinal	Standard Minimum*
Thickness, in. (mm)	0.0074 (0.188)	0.0075 (0.191)	0.0072 (0.183)
Tensile, psi (MPa)	2,002 (13.8)	2,010 (13.86)	1,200 (8.27)
Elongation (percent)	631	556	300
*Minimum values per ANSI/AWWA C105/A21.5 Standard at the time of construction.			

No damage was observed to the installed polyethylene encasement at each of the identified inspection locations. The soil samples were found to exhibit corrosive characteristics, but the exposed 8-inch diameter ductile iron pipe was found to be in excellent condition beneath the polyethylene wrap. The pipe surface displayed only minor surface oxidation. Sounding and probing of the pipe revealed no pitting or graphitization.

OBSERVATIONS & CONCLUSIONS

These investigations were performed at eight inspection sites with the cooperation of three utilities. At each site, polyethylene encasement had provided corrosion protection to ductile iron pipe in corrosive soil conditions. These results mirror numerous reports, publications, and tests that indicate polyethylene encasement has provided viable protection for millions of feet of gray and ductile iron pipe since its first use in 1958.[1,3,4,11,12,13,14,15] As with any method of corrosion protection, proper material specifications and installation procedures must be maintained to ensure the system's integrity. Appropriate national and international standards are readily available to achieve this result.[6,16,17,18,19]

Specific conclusions resulting from these investigations are as follows:

- Cell-to-cell potential surveys and side-drain technique measurements are not reliable in locating corrosion activity on polyethylene encased ductile iron pipe. During all three investigations, the potential tests indicated active corrosion where none was found.
- *In situ* four-pin soil resistivities have limited use when determining the corrosivity of soils around buried pipelines. These types of measurements cannot be taken closely parallel to the pipeline under investigation or the results can be skewed. Proper procedures for these resistivity measurements require that the pins be perpendicular to the pipeline and no closer than 15 feet from the pipe.[8] Representative soil samples from close proximity to the pipeline should always be used for more definitive evaluations of soil corrosivity in the pipe environment. Soil box resistivities measured using ASTM standard test methods[20] include compaction of and adding moisture to the sample to provide conservative results.
- Soil corrosivity tools such as the 10 point Soil Evaluation System[6] and The Design Decision Model™[7] determined that the soils at each of the investigation sites were aggressive to ductile iron pipe. These methods utilize the more conservative soil resistivity values obtained from soil box measurements of representative soil samples. Soil corrosivity tools that do not identify the method used to measure soil resistivities can be misleading and lead to inappropriate conclusions.
- At several of the inspection locations, groundwater had migrated under the polyethylene encasement but no corrosion related pitting was discovered. This supports the contention that, with time, the water trapped under the encasement is depleted of oxygen, resulting in a non-aggressive uniform environment.

SUMMARY & DISCUSSION

It has been demonstrated at BexarMet Water District in San Antonio, Texas; Charleston Water System in Charleston, South Carolina; and Onondaga County Water Authority in Syracuse, New York that polyethylene encasement is an effective system of corrosion protection for ductile iron pipe. The investigated pipelines ranged in age from 18- to 22-years, were installed in corrosive soils with polyethylene encasement for corrosion control, and exhibited no indications of corrosion-related pitting. Each of these utilities is aware of the importance of proper installation of polyethylene encasement. For that reason, they promote proper installation and educated their personnel and their contractors of the merits of the "common sense" approach of a damage free installation of polyethylene encasement.

Reliable evaluations of the condition of polyethylene-encased ductile iron pipelines cannot be achieved using over-the-line, cell-to-cell potential surveys. Soil corrosivity surveys can be a valuable tool in locating aggressive areas along the pipeline route as long as they are representative and proper soil test methods are used. The break records and/or first hand experiences of the individual utility will always be a most important tool in any assessment of polyethylene-encased ductile iron pipelines.

ACKNOWLEDGEMENTS

We would like to thank BexarMet Water District in San Antonio, Texas; Charleston Water System in Charleston, South Carolina; and Onondaga County Water Authority in Syracuse, New York for all of their assistance and cooperation with these investigations. Thanks also to Mr. Bill Foulds, P.E., and Mr. Dale Lindemuth, P.E., for their participation and consultation with much of the field work.

REFERENCES

1. T.F. Stroud, "Corrosion Control Measures for Ductile Iron Pipe", CORROSION/89, paper no. 585 (Houston, TX: NACE, 1989).
2. Bell, E.C., Romer, A.E., "Making Baggies Work for Ductile Iron Pipe", "Corrosion and Corrosion Control Research of Iron Pipe", PIPELINES 2004, ASCE Annual Conference, August 1-4, 2004, San Diego, CA
3. Ductile Iron Pipe Research Association, "Cast Iron Pipe Century and Sesquicentury Club records and correspondence", ongoing.
4. "Corrosion Protection for Ductile Iron Pipe by Polyethylene Sleeves", Kubota Ltd. Report, October 1986.
5. American Water Works Association, AWWA Engineering and Construction Division Survey, October 2000 Mainstream, Denver, CO
6. ANSI/AWWA C105/A21.5 (latest revision), "Polyethylene Encasement for Ductile Iron Pipe Systems" (New York, NY: ANSI and Denver, CO: AWWA).
7. DIPRA, "The Design Decision Model™ for Corrosion Control of Ductile Iron Pipelines", December 2004, Ductile Iron Pipe Research Association, Birmingham, AL

8. Kroon, D.H., Lindemuth, D., Sampson, S., Vincenzo, T., "Corrosion Protection of Ductile Iron Pipe", CORROSION 2004, paper no. 4046, (Houston, TX NACE)
9. A.W. Peabody, "Control of Pipeline Corrosion", National Association of Corrosion Engineers, Houston, TX, 1967.
10. Appalachian Underground Short Course, "Advanced Course" West Virginia University, Morgantown, WV, 1993.
11. A.M. Horton, "Protecting Pipe with Polyethylene Encasement, 1951-1988", AWWA Water World News 4 (1988): p.26-28.
12. Bonds, R.W., Barnard, L.M., Horton, A.M., Oliver, G.L., "Corrosion and Corrosion Control of Iron Pipe – 75 Years of Research", Journal AWWA (June 2005), AWWA Denver, CO
13. DIPRA, "Inspection Report – Cathodically Protected Ductile Iron Pipe Encased in Loose Polyethylene Film – Dickinson, ND", April 21, 2004 (Birmingham, AL)
14. DIPRA, "Inspection Report – Cathodically Protected Ductile Iron Pipe Encased in Loose Polyethylene Film – Lafourche Parish, Louisiana", May 28, 2003, Ductile Iron Pipe Research Association, Birmingham, AL
15. T.F. Stroud, "Infrastructure: Is the Problem Being Blown Out of Proportion?" Ductile Iron Pipe News, Fall/Winter 1985: p. 9.
16. ASTM A 674 (latest revision), "Polyethylene Encasement of Ductile Iron Pipe Systems" (West Conshohocken, PA: ASTM).
17. ISO 8180 (latest revision), "Ductile Iron Pipes – Polyethylene Sleeving" (Geneva, Switzerland: ISO).
18. JDPA Z 2005 (latest revision), "Polyethylene Sleeves for Corrosion Protection of Ductile Iron Pipes" (Tokyo, Japan: JDPA).
19. BS 6076 (latest revision), "Tubular Polyethylene Film for Use AS Protective Sleeving for Buried Iron Pipes and Fittings" (London, UK: BSI).
20. ASTM G 187 (latest revision), "Measurement of Soil Resistivity Using the Two-Electrode Soil Box Method" (West Conshohocken, PA: ASTM).

How to Provide Indefinite Life for Municipal Metallic Transmission Pipelines

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Abstract

Three current corrosion strategies used for Municipal Metallic Transmission Pipelines (MMTP) (defined as concrete pressure pipe, ductile iron pipe, or steel pipe 24" diameter and larger) as part of a comprehensive Asset Management Program are:

1. Run to Failure
2. Run to Failure and Repair as Required
3. Operate and Maintain to Provide an Indefinite Life (repair-free service life >100 years)

The paper will discuss the short- and long-term impacts of each strategy and detail why the strategy of "Operating and Maintaining to Indefinite Life" provides the most reliable Municipal Metallic Transmission Pipelines with the fewest repairs at the lowest total cost of operation to the owner. It will be shown that the strategies of "Run to Failure" and "Run to Failure and Repair as Required" are overall more costly since both strategies accept corrosion, unscheduled repairs, planned failure of the pipeline and the resulting high replacement costs of the MMTP. The paper will focus on methods of design, corrosion protection, maintenance and installation which allow all MMTP materials to be equally operated and maintained for an indefinite life.

Introduction

Corrosion is a significant problem for our nation. According to a 2002 Federal Highway Administration report, the annual cost of corrosion in the USA is $276 billion, with water and wastewater systems comprising $36 billion of that total. To put this close to home, consider the number of water main breaks in one's own system or in the news last year. Corrosion is a natural process that can be controlled or even prevented in its entirety. Corrosion prevention is especially critical for our high risk buried MMTP 24" diameter and larger. While corrosion in distribution systems is also an issue, this paper will focus on transmission lines 24" and larger.

To address corrosion realities, some municipalities have initiated comprehensive Asset Management strategies (Villalobos, 2006). The Villalobos paper detailed Asset Management Strategies available including Run to Failure, Run to Failure and Repair and Operate and Maintain to Provide an Indefinite Life. Distribution systems often utilize the Run to Failure or Run to Failure and Repair strategy. These strategies may be effective as the risk to the public is generally lower and the cost and complexity of unscheduled repairs to the smaller diameter pipes is manageable. However, for high risk transmission mains corrosion protection methods should be employed to provide a pipeline that has indefinite life. It will be demonstrated that this approach provides the best long term solution, maintains the public confidence in the water systems and is more cost effective than the Run to Failure strategies.

Operation and Maintenance Strategies

RUN TO FAILURE

Simply put, this strategy entails installing a transmission pipeline for the lowest initial cost and hoping for the longest possible service life. This strategy allows corrosion of the pipeline (depletion of the asset) with the expectation that the pipeline will provide a desired service life. As the pipeline corrodes, the ability for the pipe to hold internal pressures decreases as does factors of safety. This results in increasing unscheduled repairs and the possibility of lowered pumping pressures to continue operation of the line. Oftentimes, this strategy and the Run to Failure and Repair strategy are based solely on past performance history of a material. In the case of ductile iron pipe, which has significantly thinner wall thickness than cast iron pipe (as shown in Figure 1), the assumption that ductile iron pipe will provide similar service life to cast iron pipe is not warranted without additional corrosion control or protection measures.

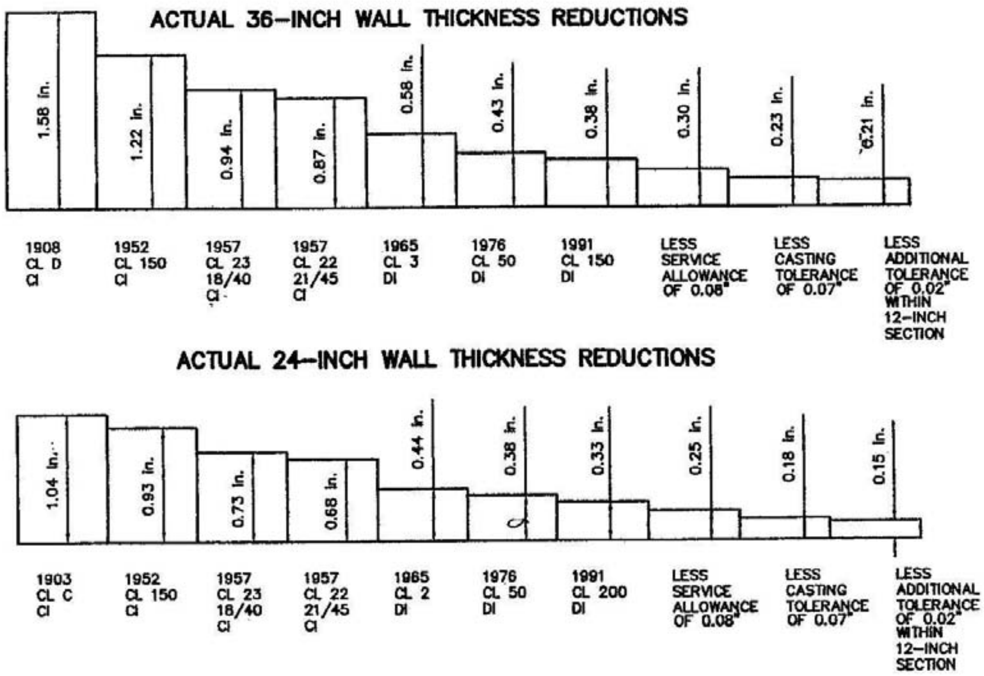

Figure 1. Actual Size of American Water Works Association (AWWA) Specification Thickness Reductions for 36- and 24-inch Diameter Cast and Ductile Iron Pipe 1908 to present (150 psi Operating Pressure) (Spickelmire, 2006)

RUN TO FAILURE AND REPAIR

Like the Run to Failure strategy, this method allows for corrosion and eventual failure of the MMTP but hopefully not before the desired design life has been achieved. At this point, another very large capital expenditure will be required to replace the original pipeline. As Villalobos and Perry report, "Run until failure and repair strategies can incorporate many of the extended life corrosion control procedures outlined in published reports and manuals… including: coatings or exterior encasements; monitoring systems; and corrosion reducing galvanic anode or impressed current systems" (Villalobos, 2006). While this method sometimes incorporates corrosion control (reducing the rate of corrosion), the MMTP will eventually fail, sometimes prior to the 50 year design life that is common in the industry. Some reports claim these corrosion control methods can extend lives to 75 years or longer but even then failure and replacement is expected. In addition, unscheduled repairs will become significant as the pipe's ability to hold internal pressure reduces due to corrosion of the MMTP.

Unscheduled Repair and Replacement Costs are significant in both of these strategies. To quantify these costs, a full life cycle cost analysis should be done for each project but as a simple example lets assume:

- Interest rate of 4.5% and Inflation rate of 3%.
- Design service life of 50 years for a 48" MMTP at initial installed cost of $5,000,000.
- Cost of repair $70,769 – Assumes two day repair, equipment, labor and significant restoration work but no claims from the public for property damage or injury (Lawrence, 2005).
- Repair costs in year 20, 30, 40 and 50 would be approximately $127,000, $171,000, $230,000 and $310,000 for each unscheduled occurrence (Lawrence, 2005).
- Replacement cost of 48" MMTP is $21,919,000 in 50 years (Lawrence, 2005).
- Villalobos and Perry state "the relative cost of the regular inspection, completion of minor repairs and cathodic protections will range between 2% and 5% of the initial capital cost of the MMTP" (Villalobos, 2006). Therefore the cost to provide a 48" MMTP with indefinite life would initially cost an additional $100,000 to $250,000, depending on the environmental conditions. There should be no significant repair costs as long as the 48" MMTP was properly designed, installed, monitored and maintained (anode replacement). It is apparent from this information that providing a MMTP with the capability of an indefinite life will be highly beneficial in both the short- and long-term. Saddling future generation with large repair and capital replacement costs of major pipelines is most likely not practicing good fiduciary responsibility. This begs the question: what must be done to have a MMTP with indefinite life in varying environments?

OPERATE AND MAINTAIN TO PROVIDE AN INDEFINITE LIFE

This strategy is forward-looking. It looks not only at present needs and initial cost but also future needs such as inspection, redundancy or maintenance of the system. Generally more planning, field survey and design work is needed. Initial decisions are key to the effectiveness of the strategy but the decisions are based on sound engineering, national standards for construction and design such as NACE (National Association of Corrosion Engineers). "The primary benefit will be a more reliable source of delivery for water or wastewater in the case of MMTPs with fewer unplanned repairs, and, in the long term, a lower total cost of operation" (Villalobos, 2006).

Steps to be Taken for MMTP to Provide an Indefinite Life

FIELD SURVEY

In areas of known corrosive activity or high risk MMTP, field surveys should be completed and a corrosion evaluation performed by an independent corrosion engineer who have no direct or indirect association with a pipe manufacturer. Steps recommended:

- Test for stray current from DC sources such as cathodic protection currents from oil and gas lines, light rail or industrial sources. Are there plans for this type of infrastructure in the future? Such plans are often unpredictable, so it is wise to make provisions to handle DC currents on transmission lines.
- Check for AC current from overhead power lines if they exist or are anticipated.
- Perform in situ resistivity and pH measurements at anticipated buried pipe elevations. In situ testing is needed as exposure to oxygen or changes in moisture content can make lab-only readings unrepresentative of the site conditions.
- Determine groundwater elevations in borings.
- Perform lab testing for chloride, sulfide, sulfate, redox potential, pH and resistivity.
- Investigate the history of pipeline corrosion in the area.
- Analyze the corrosivity of the water or wastewater and the potential for the formation of hydrogen sulfide in forcemains.

With this information an independent corrosion engineer or corrosion professional can determine if the site is in fact "corrosive" for buried pipe. For the purpose of the paper, "corrosive" soils will be defined as soils with resistivity of 5,000 ohm-cm or less (Spickelmire, 2006) and/or pH of 5 or less (Hall, 1998). Some may argue these values are conservative but for the expectation of indefinite life MMTP these types of soils should be considered corrosive and appropriate corrosion protection methods should be utilized.

STRUCTURAL DESIGN FOR INTERNAL PRESSURES AND EXTERNAL LOADS

While structural pipe design is not the topic of the paper, it does bear mention regarding both the design of MMTP as it relates to corrosivity and indefinite pipe life. All three MMTP (concrete pressure, ductile iron and steel pipe) derive their ability to hold internal pressure from either steel or iron (ferrous materials). While unprotected ferrous components assure corrosion over time, they also assure physical properties that do not change with time (unlike plastics, which have dramatic reduction in physical properties in as little as 100,000 hours or 11.4 years). Ferrous materials will not "wear out" but do corrode, resulting in less capability to hold internal pressure (lowered factors of safety) and the resulting common "pipe break." To assure the indefinite life of the pipeline, designers should consider engineering pipelines for working and surge pressures higher than those initially expected to account for corrosion among other issues. In a previous paper it was recommended that all transmission lines be designed for no less than 150 psi working pressure with appropriate addition for surge so no "weak links" will exist and the system will have the ability to handle higher pressures for future demand (Mielke, 2004). Surge has a significant impact on MMTP longevity and especially for concrete pressure pipes where under-design or corrosion of reinforcement wire can result in cracking of the cement-mortar coating and/or over stressed reinforcing wire. The author has seen a trend toward higher calculated design surge pressures and even design of pipelines to pump shut off head to minimize risk.

CORROSION PROTECTION METHODS FOR THE INTERIOR OF MMTP

Conveyance of raw and treated water is the most common task for MMTP. Cement mortar or concrete is the most common corrosion protection measure used for lining MMTP. Upon application of the high pH cement a passivating iron oxide forms which protects the ferrous components from corrosion. The passivating film is maintained by the high pH environment and has been proven to be an excellent lining material which prevents both pipe wall corrosion and tuberculation. It is recommended that one "fill interior joint recesses with cement mortar" after joint completion to assure formation of the oxide layer at the joint (Hall, 1998). For corrosive water (pH of 5.5 or less, or containing chemicals corrosive to concrete such as sulfates or chlorides), barrier type linings such as PVC (Hall, 1998), epoxy or polyurethane should be used. In forcemains where the line runs continually full and no air or air pockets exist cement-mortar lining is commonly used.

For gravity sewers or forcemains where air can enter the line or be trapped, such as at high spots or partial flow areas, cement-mortar lining is not the best option. Air allows for the breakdown of the sewage producing hydrogen sulfide gas, which combines with humidity on the crown of the pipe wall, creating sulfuric acid. This acid will attack the cement-mortar lining and render the passivating iron oxide film useless, resulting in direct corrosion of the unprotected ferrous components. In these environments, barrier coatings such as epoxy or polyurethane are recommended for ductile iron or steel pipe and PVC liners for concrete pipe. A key element for the barrier coatings is the surface preparation (generally sand blasting or similar) required for good adhesion to the pipe wall. Without good surface preparation and the resulting adhesion creating a "tight bond," the effectiveness of these coatings is minimal.

For pipelines expected to operate well in excess of 100 years, provisions should be considered in design for future pigging or cleaning of the line. Incorporation of shop fabricated wyes will allow access in the future for cleaning. Reduced head loss from cleaning of pipelines can be cost effective, improve water quality and provide additional longevity for cement-mortar lining. Shop installed manways should always be incorporated into the design to allow internal inspection of the pipeline and access points for leak detection systems if the need arises in the future. Adding these features during design is straight forward, cost effective and eliminates the need for large field taps in the future.

Corrosion Control Methods in Non-corrosive Environment

MMTP and their ferrous components are not corrosive in all soils. In neutral, free draining soils, those described with uniform resistivity in excess of 5,000 ohm-cm, pH of greater than 5, redox >100 mv, no stray DC currents and no significant chloride, sulfates or sulfides concentrations, an indefinite life is possible. In the "right" environment, cast iron pipe in France has been shown to provide service since the 1600's (Villalobos, 2006). Figure 2 details a 1906 steel pipe line in NJ being extended by welding to new tape coated steel pipe for the next 100 years of service. The difficulty with most projects is assuring that there won't be sections or pockets of soils that are indeed "corrosive" since soils are generally not homogeneous, especially over the route of a long transmission main. Future stray currents are also hard to predict, along with the impact of road salts or heavy fertilization.

Figure 2. 1906 72” Steel Pipe connected to new Steel Pipe in NJ (National Welding Corporation, 2006)

Corrosion monitoring is an economical means of addressing the “what-if’s” of MMTP in non-corrosive soils. The system consists of bonded joints (provides electrical continuity across ductile iron gasketed joints, as shown in Figure 3), insulated joints at connections to existing pipe or dissimilar pipe material, and monitoring test stations. The net result of this inexpensive corrosion monitoring system is that it allows the owner to monitor the pipeline for corrosion in the future and be proactive by taking corrective measures, such as installing buried anodes, long before a pipe break occurs.

Figure 3. Ductile Iron Pipe Bonded Joint (Southwest Pipeline, 2004)

Some discourage making a pipeline (electrically) continuous and instead prefer to deal with corrosion in isolated sections of pipe. Others would suggest that by making the pipeline continuous, corrosion will take place over a much larger surface area and as such will minimize the impact of the corrosive area. It can be argued that continuous pipe does have a positive impact as virtually all ferrous pipelines were electrically continuous prior to approximately 1960 due to the methods of jointing. Lead joints on iron pipe and riveted or welded joints for steel or concrete pipe produced continuous pipe by default. Rubber gasketed push joints for ductile iron pipe (developed in late 1950's) (Bonds, 2003) or steel or concrete pipe effectively insulate between pipe joints as long as there is no metal-to-metal contact between pipe ends. Since all MMTP materials utilize gasketed joints, joint bonding should be a standard practice.
Monitoring systems are very cost effective, even for owners that choose to practice Run to Failure or Run to Failure and Repair strategy. Monitoring systems typically cost less than 1% of the initial cost of the project. The corrosion monitoring systems will provide a "window" to determine if corrosion is present in specific areas of the pipeline and since all the "electrical" connections were completed during initial construction, the cost of corrective measures to address problem areas will be relatively small. Maintenance budget overruns and the impacts on the public can be minimized. Both the concrete pressure pipe industry (Hall, 1998), (ACPPA) (Prosser, 2003) and the steel pipe industry (Northwest Pipe, 2006) recommended bonded joints, test stations and monitoring of pipelines as good practice.

CONCRETE PRESSURE PIPE IN NON-CORROSIVE ENVIRONMENT

High pH mortar in contact with the steel cylinder and reinforcing wire or bar forms a passive iron oxide film. This oxide film is maintained by the alkalinity of the cement-mortar coating. As long as the exterior joint recess is properly filled and cured with cement mortar and movement from mechanical restraints does not result in cracking of the cement-mortar coating, indefinite life can be achieved in this environment. To accomplish this for prestressed concrete cylinder pipe, specify both "bonding plates" for joint bonding connections and "shorting straps," each of which are cast into the pipe wall. Shorting straps reduce the voltage drop in the long prestressing wires if cathodic protection is provided (Hall, 1998). Joints should be bonded to the bonding plates and monitoring systems installed and monitored periodically.

DUCTILE IRON IN NON-CORROSIVE ENVIRONMENT

Ductile iron depends primarily on wall thickness to provide desired service life. In non-corrosive environments where the corrosion rate is measured in mils/year is very low standard ductile iron pipe may be able to provide indefinite service life. It still would not be appropriate to equate the expected life of ductile iron to cast iron pipe as the thickness of ductile iron pipe "can be as much as 75% thinner for a similar pressure and diameter pipe" (Spickelmire, 2006). There is much debate on the use of polyethylene encasement to address this dramatic decrease in wall thickness. When bonded joints (Figure 3) are used and monitoring stations installed, the polyethylene encasement may actually cause problems.

At best, it is difficult to install polyethylene encasement without damage (hole, tears, joint leaks etc.). Also problematic is the fact that polyethylene encasement is loosely attached to the pipe wall and not considered a bonded coating per NACE Standard RP0169. Bonded coatings do not allow corrosion to proceed laterally past an area of damaged coating. If monitoring determines that cathodic protection is needed in an area, the polyethylene encasement can effectively "shield" the pipe wall from protective cathodic currents that are not directly in contact with the soil. Since the polyethylene encasement allows corrosion to advance laterally past the damaged area and the polyethylene encasement shields the pipe wall from protective cathodic currents, corrosion is most likely to continue under the polyethylene encasement. As evidence, the Ductile Iron Pipe Research Association recommends the use of polyethylene encasement when crossing oil or gas lines with cathodic protection to "shield" the pipe from the cathodic protection currents that would like to "consume" the ductile iron pipe to protect the oil or gas line (Bonds, 1997). Polywrap shielding in essence may not allow the ductile iron pipeline to be properly monitored or cathodically protected. The author is aware of no national standard for the design of cathodic protection on polyethylene wrapped ductile or steel pipe.

STEEL PIPE IN NON-CORROSIVE ENVIRONMENT

Steel pipe offers a number of AWWA coating systems that can provide indefinite life in most environments. The most common are three-layer tape system, polyurethane, cement mortar, extruded polyethylene and coal-tar coatings. Polywrap is not recommended for steel pipe due to previously discussed installation issues and the fact it is not tightly bonded to the pipe.

The cement-mortar coating option is similar to the coating system used on concrete pressure pipe. As previously mentioned, cement-mortar coatings can provide an indefinite life in non-corrosive soils that are free from stray currents.

The balance of the coatings is bonded dielectric coatings, which act as a barrier coating and provide excellent dielectric resistance to stray currents. Per NACE Standard RP0169 bonded coatings depend on good surface preparations and the resulting good adhesion strength to provide corrosion protection. The adhesion strength allows the coating to stay tightly bonded to the pipe wall surface during all phases of construction and not allow the migration of water or air between the coating and pipe wall to begin a corrosion cell. Dielectric coatings are tough and resist construction damage with thickness of up to 80 mils for tape systems versus 8 or 4 mils for polyethylene encasement. Still holidays may occur in the coatings that could cause pitting corrosion in pockets of corrosive soils. Monitoring systems are recommended to address this "what if." In the future, if areas of excess current flow (corrosion) are detected during periodic monitoring, cathodic protection using buried anodes could be simply installed at that time without "digging up" sections of the pipeline. With the tightly bonded coating the cathodic protection currents would be assured to protect the areas of damaged coatings. This process is very similar to that used successfully on oil and gas pipelines throughout the USA and by virtually all independent corrosion engineers using the NACE Standard RP0169. Simply put, it works. As evidence, compare the number of gas leaks in your community to the number of water leaks.

Corrosion Protection Methods in Corrosive Environment

In corrosive environments, corrosion protection methods, which stop or prevent corrosion, in conjunction with cathodic protection should be used for all MMTP to provide indefinite life. Corrosion control methods accept corrosion of the asset, unscheduled repairs and large replacement costs and are not appropriate for the "Operate and Maintain for Indefinite Life Strategy" in corrosive environments. As such, the balance of the paper will address MMTP corrosion protection options for owners and engineers.

CONCRETE PIPE IN CORROSIVE ENVIRONMENT

Cement-mortar coatings are susceptible to the loss of their corrosion protection ability in corrosive environments, both from soil and stray currents. S.C. Hall describes certain conditions and a sampling of possible solutions follows (Hall, 1998):

- Sulfate soils with more than 0.2% SO_4^{2-} or waters containing more than 2000 SO_4^{2-} may require Portland cement with 5% tricalcium aluminate.
- Acid soils with pH less than 5.0 may require the exterior to be coated with high build coal-tar epoxy.
- High chloride soils of greater than 350 ppm may require the exterior to be coated with coal-tar epoxy.
- Stray current electrolysis caused by discharge from cathodic protection system or other DC sources may also need to be coated with high build coal-tar epoxy.
- Subaqueous or high ground water installations may require high build coal-tar epoxy coating of the pipe and the joint (Carnegie) rings.
- Protection of steel joint rings requires epoxy or zinc coating in addition to filling of joint recess with cement. As commentary by the author, movement at restrained joints should not be allowed during field test pressures or surge events to limit cracking of the cement-mortar coating at he joints and possible corrosion. Typically mechanical restrained joints allow longitudinal and/or rotational movement as they take up slack or engage with thrust loads. Welded restrained joints should be considered to limit joint movement.
- Application of cathodic protection requires the use of shorting straps for PCCP, bonded joints and test station leads previously discussed to provide corrosion monitoring. The cathodic protection headers are integral with the cathodic protection system and can consist of galvanic (buried anodes bags or ribbon anodes comprised of zinc or magnesium) or impressed current system.

DUCTILE IRON PIPE IN CORROSIVE ENVIRONMENTS

In order to provide indefinite life in corrosive environments, ductile iron pipe requires corrosion protection systems and design per NACE standard RP0169. Tightly bonded dielectric coatings offer an effective solution and history, as they provide both a barrier coating that is resistant to virtually all soil corrosivity issues as well as resistance to stray currents. Possible corrosion protection options include:

- Polyurethane coating such as US Pipe Polythane™, a product that was produced and sold in USA beginning in 1988 (Horton, 1995). A. M. Horton reports that Polyurethane "cures quickly to form a hard, yet flexible film that is resistant to chipping, cracking and impact damage." Horton also reports that on one 31,000' project shipped over 2,000 miles, "the exterior coating had little or no shipping damage and had an installed coating efficiency of 99.66% when tested as part of a cathodic protection system (Madison Chemical, 1994)" (Horton, 1995).

Figure 4. Ductile Iron Pipe with polyurethane coating (Szeliga/Lieu, 2002)

- Coal-tar epoxy coating. A.M. Horton reports "coal-tar epoxy has been used to protect the interior and exterior of iron pipe in excess of 40 years. It has been proven to be an excellent protective coating if properly applied in sufficient thickness" (Horton, 1995).
- Tapewrap coating. Available in two- (50mil) or three- (80 mil) layer systems from tape manufacturers specifically for ductile iron pipe. The tape product is also available in a system that requires no sand blasting and can utilize standard factory shop-coated pipe for tapewrap.

Figure 5. Tape-coated Ductile Iron Pipe (Szeliga/Lieu, 2002)

- Extruded polyethylene, such as ShawCor Pipe Protection's Pritec™ system. Similar to tape system except the coating is extruded onto the pipe to form a tough 70 mil coating.

- Surface preparation for ductile iron pipe per NAPF (National Association of Pipe Fabricators) 500-03. All of the above coatings, with the exception of one tape system option, require a prepared (generally blasted) surface for proper bonding of the coating to the surface. The NAPF-500 national standard was developed by pipe manufactures, consulting engineers, coating suppliers and fabricators for surface preparations of ductile iron pipe and fittings. Surface preparation is an ongoing essential component for pipe lining application and coating of fittings. Currently there is much controversy on the ability to properly blast and/or prepare the exterior surfaces of ductile iron pipe for coating application in the USA despite the NAPF 500 standard and the apparent successful coating history. It has been argued that the recent refusal of the USA ductile iron pipe industry to provide exterior bonded coatings is driven by the high selling price of the coated product (Spickelmire, 2006).
- Zinc-rich coating with a top coating has been used over the past 30 years in Europe per ISO Standard 8179. Some USA ductile iron pipe manufacturers have and are most likely manufacturing the zinc-coated product in the USA and shipping overseas (ACIPCO International, Feb. 7, 2007).
- Zinc-AL coating with blue epoxy finish coat produced and marketed as PAM Natural by Saint-Gobain, the largest ductile iron pipe manufacturer in the world. (Saint-Gobain, Dec. 20, 2006). Product is represented as more durable than Zinc-rich coating and manufactured per British Standard EN 545:2002.
- External coatings referenced in the British Standard EN 545:2002 include:
    1. Zinc rich paint coating having a minimum, mass of 150 g/m2, with finishing layer,
    2. Thicker zinc coating, having a minimum. mass of 200 g/m2, with finishing layer,
    3. Polyethylene sleeving (as a supplement to the zinc coating with finishing layer),
    4. Zinc-aluminum (85Zn – 15Al) coating having a minimum. mass of 400 g/m2, with finishing layer (PAM Natural),
    5. Extruded polyethylene coating,
    6. Polyurethane coating,
    7. Fiber reinforced cement mortar coating having a nominal thickness of at least 5mm, and
    8. Adhesive tape.
- Other European Coating standards for ductile iron pipe include:
    1. DIN 30 675 Part 2 “Corrosion protection systems for ductile iron pipes,”
    2. DIN 30 674-3 “Coating of ductile cast iron pipes. Part 3: Zinc coating with a protective finishing layer,”
    3. DIN 30 672 “Tape and shrinkable materials for the corrosion protection of buried and underwater pipelines without cathodic protection for use at temperatures up to 50C,
    4. DIN 30 6704 Part 2 “Cement mortar coatings for ductile iron pipe,” and
    5. EN 14628 Ductile Iron pipes, fittings and accessories – External polyethylene coating for pipes – requirements and test methods. (This is not polyethylene encasement).

- Polyethylene encasement is a loose or unbonded coating and NACE RP0169 recommends against the use of loose or unbonded coatings. Much discussion and controversy regarding the use of polyethylene encasement in general but especially on transmission lines with cathodic protection. Installation damage and the fact the polywrap is not tightly bonded to the pipe is key. As Spickelmire reports, "there are no industry standards for cathodic protection of polyethylene-encased ductile iron pipe… The major problem is that no long-term non-biased scientific study shows whether polyethylene encasement with cathodic protection works or not" (Spickelmire, 2006). Szeliga also reports that based on "actual experiences of independent (owners or their consultants)" that "PE encasement is not adequate for corrosion control of DI pipe in corrosive soil if the risk of pipe failure in not acceptable" (Szeliga, 2007).
- Application of cathodic protection includes bonding of joints, heat shrink sleeves at joints, electrical isolation from other pipelines, installation of monitoring stations and either galvanic (magnesium anodes) or impressed current cathodic protection in accordance with RP0169.

STEEL PIPE IN CORROSIVE ENVIRONMENTS

Like both concrete and ductile iron, steel requires corrosion protection methods in conjunction with cathodic protection to provide indefinite service life. Steel water pipe has extensive experience in providing corrosion protection measures as the industry has used much of the knowledge, standards and practices from the oil and gas industry. The result is in general a good long-term performance history. Corrosion protection options for buried pipe include:

- Tapewrap system per AWWA C214 and C209
- Polyurethane per AWWA C222
- Epoxy per AWWA C210
- Extruded polyolefin system per AWWA C215
- Coal-tar enamel per AWWA C203
- Heat shrink sleeves per AWWA C216. Used to complete joint for all dielectric coated pipelines.
- Surface preparation specifications are included in the AWWA spec and reference SSPC standards.
- Cement mortar per AWWA C205. This is a similar standard to the cement-mortar coating in the AWWA standards for concrete pressure pipe. Environmental limitations for cement-mortar coatings are mentioned in the concrete pressure pipe section.
- Application of cathodic protection includes bonding of joints, heat shrink sleeves at joints, electrical isolation from other pipelines, installation of monitoring stations and either galvanic (magnesium anodes) or impressed current cathodic protection per NACE RP0169 standard.

Conclusions

Transmission pipelines are very expensive to build and come with high risk due to the everyday dependence by the public and the risk of injury and or property damage from failures. The cost of unscheduled repairs and enormous replacement costs of transmission pipelines can result in undo financial burdens. In the early to mid-1900's, owners were using the best pipe materials and practices available at the time and hoped for the best. Some of these pipelines have performed quite well, perhaps due to non-corrosive soils. Today owners, engineers and manufacturers have a much broader technology base but there is scattered understanding of the process of corrosion control and corrosion protection. Some of this is likely due to lack of exposure, reluctance to change and commercial issues. In any regard, it is recommended that some asset management strategies be employed when designing or building transmission pipelines. It can be concluded that Operating and Maintaining to Provide Indefinite Life is the best strategy as it assures long term service with the lowest overall cost and fewest unscheduled repairs. The knowledge, technology and products exist to provide an owner with water transmission pipelines that can be constructed, operated and maintained for indefinite life.

Recommendations

1. Conduct a field survey as detailed in the paper. If the environmental conditions are unknown, how can informed decisions be made regarding corrosion strategies?
2. Operate and Maintain to Provide Indefinite Life is the best corrosion strategy for transmission lines and provides the lowest overall cost and fewest unscheduled repairs. Use corrosion prevention practices as they stop or eliminate corrosion.
3. Utilize Equal Corrosion performance specifications for all MMTP materials. If a corrosion protection strategy is selected, then all MMTP materials should require a similar level of corrosion performance (corrosion protection).
4. Always bond joints and provide monitoring systems for all MMTP. This practice is inexpensive when done during construction and provides a "window" to monitor the pipeline. Monitoring for the first few years can be supplied by the pipe supplier if the owner is unfamiliar or lacks the personnel.
5. Specify the corrosion protection levels needed for transmission pipeline systems. Don't be satisfied with the status quo or "this is what is available" when it comes to critical corrosion decisions. Our dependents are counting on us.

References

ACIPCO International website, "Intro- Coatings," www.acipco.com/international/liningsandcoatings/coatings/intro.cfm (Feb. 7, 2007).

Bonds, Richard W, P.E. (2003). "Ductile Iron Pipe Joints and Their Uses" DIPRA DIPJ.4-03 Published 4-03.

Bonds, Richard W. P.E. (1997). "Stray Current Effects", DIPRA Publication, SC/6.03/2.5M, Published 8-97.

British Standard BS EN 545:2002, Ductile Iron pipes, fittings and accessories and their joints for water pipelines- Requirements and test methods.

Hall, PE, Sylvia C. (1998). "Corrosion Control of Prestressed Concrete Cylinder Pipe", ASCE Pipelines in the Constructed Environment, August 1998, San Diego CA

Horton, PE, A.M. (1995). "Special Protective Coatings and Linings for Ductile Iron Pipe", ASCE Pipeline Advances in Underground Pipeline Engineering, June 1995, Bellevue WA.

ISO Standard 8179 (2004 Edition). "Ductile iron pipes – External Zinc Coatings"

Lawrence, PE, Charles F., (2005). "Economics of Cathodic Protection", Appalachian Underground Corrosion Short Course, May 2005, Morgantown WV and spreadsheet.

Mielke, Richard, P.E. (2004). "A Guide for the Design of Water Transmission Pipelines", ASCE Pipeline Conference, August 2004, San Diego CA.

NACE (National Association of Corrosion Engineers) Standard Recommended Practice RP0169-2002 "Control of External Corrosion on Underground or Submerged Metallic Piping Systems".

NAPF (National Association of Pipe Fabricators) 500-03 "Surface Preparation Standard for Ductile Iron Pipe and fittings receiving special external coatings and/or special internal linings". March 2000

Northwest Pipe Suggested Specification – Steel Pipe for Water Transmission. Cement-mortar lined and Tape Coated Rev 01-2005.

Prosser, D.P., (2003). "Concrete Pipe for the 21st Century," American Concrete Pressure Pipe Association.

Saint - Gobain website: "Business Sector - Pipe" http://www.saint-gobain.com/en/html/groupe/canalisation.asp> (Dec 20, 2006).

Saint - Gobain website: "Pipe and Fittings: Pam Natural Advantages", http://www.saint-gobain-pipelines.co.uk/watersewer/pipes/naturalstandards.cfm> (Dec. 20, 2006).

Spickelmire, William S. (2006). "Ductile Iron Corrosion Control Design Considerations and Risk Assessment," CA-NV-AWWA Spring Conference, April 2006 Burlingame CA.

Szeliga, Michael J, and Lieu, Don (2002). "Protecting Underground Assets with State-of-the-Art Corrosion Control," Materials Performance Magazine July 2002

Szeliga, Michael J, P.E. (2006). "Corrosion and Corrosion Control for Trenchless Applications," North Carolina AWWA-WEA December 2006, Statesville NC.

Szeliga, Michael J, (2007). "Analysis of Ductile Iron Corrosion Data from Operating Mains", Materials Performance Magazine February 2007.

Villalobos, PE. Jose L., Perry, Carl (2006). "Asset Management Strategies for Metallic Municipal Transmission Pipelines," ASCE Pipeline Conference, July 2006, Chicago IL.

Challenges of Disposing of Tunneling Water

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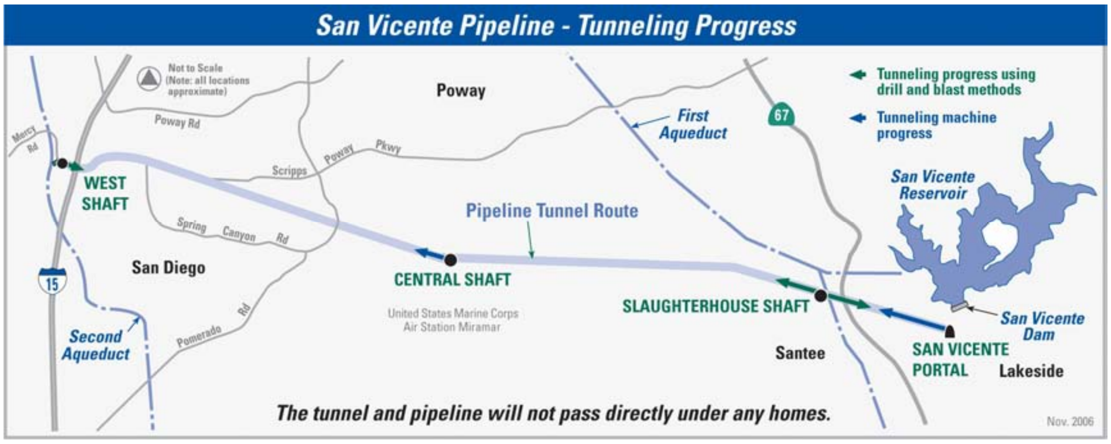
Abstract

The San Diego County Water Authority (Water Authority) is currently constructing the San Vicente Pipeline connecting the San Vicente Reservoir to the Second Aqueduct Pipeline. This project is a key component of the Water Authority's Emergency Storage Program (ESP) to provide greater water reliability in the Water Authority's delivery system. The San Vicente Pipeline is approximately 11-miles in length and is being constructed as a 12.5 to 14-foot (3.81 to 4.27-meter) diameter tunnel through varying geology. The depth of the tunnel varies from 50 feet (15.24 meters) to over 500 feet (152.4 meters) below ground surface. There are four access sites to the tunnel. Each of these access sites has a unique set of parameters, site restrictions and expected groundwater inflows.

It is difficult to predict the volume of groundwater that will be encountered, but 1000 gallons per minute is the upper limit in the project's Geotechnical Baseline Report. In addition, the current discharge regulations require high levels of groundwater treatment for discharge to surface waters. The majority of the groundwater that is currently being encountered is being treated and discharged to a local streambed. The cost of the treatment of groundwater is expensive and has led the Water Authority to look for other cost effective means of disposing groundwater. This paper addresses the other means of disposal and the various processes such as permit, community, and contract requirements that govern their implementation.

Background

The San Diego County Water Authority (Water Authority) is currently constructing the San Vicente Pipeline connecting the San Vicente Reservoir to Second Aqueduct Pipeline. This project is a key component of the Water Authority's Emergency Storage Program (ESP) to provide greater reliability in the Water Authority's delivery system. The San Vicente Pipeline is approximately 11-miles (17.69 kilometers) in length and is being constructed as a 12.5 to 14-foot (3.81 to 4.27-meter) diameter tunnel through varying geology. There are four access sites to the tunnel. Figure 1 shows the major features of the project.



The San Vicente Pipeline extends from the Water Authority's Second Aqueduct on the west, near the intersection of Interstate 15 and Mercy Road, to the San Vicente reservoir on the east. The tunnel excavation consists of three vertical access shafts; West Shaft, Central Shaft and Slaughterhouse Shaft, and a horizontal access point, the San Vicente Portal, located near the base of the existing San Vicente Dam and Reservoir. The tunnel is being excavated using various techniques, including drill and blast, a main beam tunnel boring machine (TBM), and two digger shields. The tunnel has a high point at Central Shaft and slope downward to the West Shaft and the San Vicente Portal. The depth of the tunnel varies from 50 feet (15.24 meters) to over 500 feet (152.4 meters) below ground surface.

Groundwater

Subsurface geotechnical data was collected as part of the project design. A number of piezometers were placed during the boring operation to monitor the groundwater levels throughout construction. The groundwater is quite deep throughout most of the tunnel alignment, with groundwater expected in the most westerly 15,000 feet (4,572 meters) and the most easterly 28,000 feet (8,534 meters) of tunnel construction. The westerly portion of the tunnel drains to the West Shaft and the easterly portion of the tunnel drains to Slaughterhouse Shaft and the San Vicente Portal. All of the westerly portion of the tunnel will eventually drain to the San Vicente Portal when the breakthrough occurs between the Slaughterhouse Shaft and the San Vicente Portal. It is difficult to predict the volume of groundwater that will be encountered, but 500 gallons per minute (31.5 liters per second) is the upper limit with 1,000 gallons per minute (63.1 liters per second) being anticipated at the San Vicente Portal after the breakthrough to Slaughterhouse Shaft. The Geotechnical Baseline Report (GBR) estimated the flow rates at the four sites as follows:

Table 1
Anticipated Groundwater Inflows

Site	GBR Maximum Flow Rate
West Shaft	500 gpm (31.5 liters per second)
Central Shaft	500 gpm (31.5 liters per second)
Slaughterhouse Shaft	500 gpm (31.5 liters per second)
San Vicente Portal	1,000 gpm (63.1 liters per second)

As part of the baseline geotechnical work for the project, samples of the groundwater were taken at the 12 boreholes and analyzed. Prior to collecting the samples the piezometers and boreholes were purged. In addition, two private wells were sampled, one in Beeler Canyon between Slaughterhouse Shaft and Central Shaft and one in Slaughterhouse Canyon.

Temperature of the groundwater is between 68 and 75 degrees F (20 and 23.9 degrees Celsius). A general summary of the groundwater quality, based on data from the borings and piezometers is presented in Table 2. The water quality varies considerably from well to well. The groundwater can be classified as very hard and mineralized. From a mineral water quality standpoint, the poorest water quality is from a well in Slaughterhouse Canyon.

Table 2
General Groundwater Quality[1]

Parameter	Range of Values
pH	6.18 - 7.72
TDS, mg/L	260 - 3190
Hardness, mg/L as $CaCO_3$	182 - 2030
Sodium, mg/L	115 - 323
Sulfate, mg/L	50 - 617
Chloride, mg/L	120 - 1510
Nitrate-N, mg/L	Non-detect – 2.53
Fluoride, mg/L	0.37 - 3.2
Iron, mg/L	0.8 - 19
Manganese, mg/L	0.18 - 5.65
Copper, mg/L	Non-detect – 0.06
Zinc, mg/L	Non-detect – 1.15
Lead, mg/L	Non-detect – 0.057
Total Chromium, mg/L	Non-detect – 0.028
Trace Organics EPA 524.2	Non-detect
Trace Organics EPA 525.2	Non-detect

[1] From Geotechnical Data Report, Dec. 17, 2004, Appendix A11

It is important to note that the water quality represented in Table 2 is only generally indicative of the quality of water, which may be encountered, in the tunneling operations. The water quality is expected to vary from location to location as the tunneling progresses.

Alternatives for Disposal of Groundwater

Knowing the amount of groundwater to be expected at each of the four sites, the initial step was to develop alternatives for the disposal of groundwater for each site. Each site was treated separately because of unique conditions at each site. Two of the sites, West Shaft and Central Shaft, are located within developed areas of the City of San Diego. The other two sites, Slaughterhouse Shaft and the San Vicente Portal, are located in rural areas of San Diego County without much surrounding infrastructure or development. The various alternative disposal methods identified included:

- Discharge to Surface
- Irrigate Adjacent Land
- Discharge to City of San Diego Sewer System
- Discharge to Water Authority's Aqueduct System
- Discharge to City of San Diego San Vicente Reservoir

These disposal methods were categorized as either primary or back up. Primary systems are able handle the maximum estimated flow. Secondary systems are viable but have capacity limitations.

The next step was to create a matrix for each site that identified the significant issues and limitations of each alternative. Estimated costs were also included when available. The matrix was used to evaluate the viability of each alternative on a site-by-site basis.

Discharge to Surface. The "discharge to surface" alternative initially included two separate regulatory strategies:

1. Comply with the existing General Waste Discharge Requirements established for groundwater extraction discharges in the region, or

2. Obtain and comply with an individual National Pollutant Discharge Elimination System (NPDES) permit established for this project

The first strategy was envisioned during the design phase and referenced in the contract documents. Inadvertently, compliance with an outdated permit was specified in the contract. The current permit, adopted in 2001 by the California Regional Water Quality Control Board (Regional Board), had more stringent standards than the one specified. As a result, the groundwater from the tunnel requires treatment to remove metals before it can be discharged to surface waters. This groundwater treatment is a changed condition to the contract.

In an attempt to avoid the costly treatment of groundwater, the second "discharge to surface" strategy was also pursued. The contractor submitted the individual permit application to the Regional Board six months after construction started. The permit application was abandoned several months later. It became apparent that the permit application would not be processed in a timely manner by the regulatory agency and the permit conditions would likely be similar to the existing general permit for groundwater extraction discharges. The Regional Board does not have the flexibility to deviate from the California Toxics Rule standards.

Irrigate Adjacent Land. During the initial low flow of groundwater, the contractor used the groundwater for dust control. When flows increased, the contractor installed an irrigation system to irrigate the adjacent vacant land, which handled flows up to 30 gallons per minute (1.9 liters per second). Permission from the adjacent property owners was required in some instances. The City of San Diego required the Water Authority to monitor and mitigate for the new vegetation growth on their open space. In addition, the contractor monitored the sites for surface runoff and disengages the irrigation system when the ground is saturated and during rain.

Discharge to the City of San Diego Sewer System. The City of San Diego sewer mains are in the proximity of the West Shaft and Central Shaft. A City of San Diego ordinance allows the discharge of groundwater into its sewer system on a temporary basis. The discharge must comply with the conditions of an industrial waste discharge permit issued by the City's Metropolitan Wastewater Department. The discharge rates are based upon water quality, Total Suspended Solids (TSS) and Biological Oxygen Demand (BOD). The lowest discharge rate of $2.50 per 100 cubic feet (2, 831.7 liters) applies. Based on the current groundwater inflows of 15 to 25 gallons per minute (0.9 to 1.6 liters per second) at the West Shaft, the average monthly billing from the City of San Diego for discharge into the sewer system is $2,500. This is substantially more cost effective than the contractor's proposed ion exchange treatment method at this site.

Discharge to Water Authority's Aqueduct System. At the West Shaft and Slaughterhouse Shaft, the Water Authority's aqueduct or distribution pipelines are in the proximity. The pipelines convey untreated – blended Colorado River and State Project water. The water from the tunneling operation will be significantly diluted with the aqueduct water. The flows in Pipeline 5 of the Second Aqueduct, the discharge point for the West Shaft, are fairly consistent and range from 280 to 320 cubic feet per second (7,928.7 to 9,061.4 liters per second). The flows for the First Aqueduct, the discharge point for the Slaughterhouse Shaft, range from 20 to 50 cubic feet per second (566.3 to 1,415.8 liters per second). Based upon this, the minimum dilutions for the West and Slaughterhouse Shafts discharges are approximately 280:1 and 20:1 respectively, based upon 500 gallons per minute (31.5 liters per second) of treated tunnel water. Even the minimum dilution (20:1) will be adequate to mitigate most of the chemical constituents of concern to insignificance. Disposal of treated groundwater into the aqueduct is desirable because the groundwater would no longer be wasted. The groundwater would become an asset as part of the untreated water supply to the San Diego region. Untreated water supply generates revenue for the Water Authority. The California Department of Health Services (DHS) regulates water supply sources so DHS approval is required and is currently being pursued.

Discharge to City of San Diego San Vicente Reservoir. The San Vicente Portal is located at the base of the San Vicente Dam. It is physically possible to pump water from the tunneling operation into the San Vicente Reservoir. The San Vicente Reservoir is owned by the City of San Diego and their permission is required to implement this alternative. Initial talks with the city indicate they are receptive to this concept. The Water Authority is currently collecting the water quality data to present to the city as part of the approval process. In addition, as part of the ESP, the Water Authority intends to raise the existing San Vicente Dam. The reservoir will need to be lowered to accommodate the construction activity associated with the dam raise. The Water Authority is investigating any impact the discharging of tunnel water into the reservoir may have on the future Dam Raise project.

Summary

The Water Authority is actively looking for alternative and cost effective means of disposing of groundwater from the San Vicente Pipeline for two reasons:

1) The volume of groundwater that will be encountered in constructing the tunnel is difficult to predict, and

2) The current regulations require high levels of groundwater treatment for discharge to surface waters, which is expensive.

The Water Authority identified five alternative disposal methods. The discharge to surface alternative was abandoned because it proved to be too time consuming and unlikely to result in discharge requirements different than the ones currently being implemented. The alternatives of using the groundwater to irrigate adjacent land and discharge to the city of San Diego sewer system have been implemented. The Water Authority is currently collecting water quality data of the tunnel water to perform an analysis on the viability and effectiveness of discharging to either the Water Authority's Aqueduct system or the city of San Diego San Vicente Reservoir.

PILOT TUBE MICROTUNNELING EXPLODES IN THE U.S. USING VITRIFIED CLAY JACKING PIPE

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ABSTRACT:

First introduced in the United States in 1995, Pilot Tube Microtunneling (PTMT) has been increasing in popularity year after year. This trenchless method of installing sewer pipe is essentially a hybrid of three trenchless boring techniques:

1- Having a slant faced steering head similar to that of a directional drill,
2- Utilizing the guided accuracy of a conventional microtunnel machine,
3- Using an auger type spoil removal system similar to a horizontal bore.

Among the reasons for the popularity of this system are; low equipment costs, relatively small topside footprint, and small jacking pits. Each year more contractors are purchasing these inexpensive and easy to operate tunneling machines from three equipment manufacturers: Akkerman, BohrTec, and Wirth-Soltau.

Initially, pipe sizes ranged from 4 inch to 12 inch with maximum drive lengths up to 250 feet. Currently, the largest diameter (in the U.S.) installed by the pilot tube microtunnel method is 27 inch I.D. /32 inch O.D. and maximum drive lengths are now just over 400 feet. Larger diameters and longer drive lengths are due to the development of better optics in the guidance system and more powerful hydraulics in the jacking frame.

This paper will explain the method of how sewers are constructed using PTMT in detail (two pass and three pass methods), as well as briefly discuss in a case study manner, showing reasons the installation method was chosen, project statistics, and difficulties or challenges of numerous completed and current vitrified clay pipe pilot tube microtunnel projects.

1. ORIGINS

The pilot tube method of microtunneling originated in Japan and Europe nearly two decades ago as a way to install 4 and 6 inch house connections using trenchless techniques. Today, this technology has grown to installations with pipe diameters up to 1200 mm (48 inches) (in Europe) and drive lengths in the 400 LF range. The primary reason for this growth is the achievement of the same accurate on-line and on-grade installation as conventional microtunneling, but with significantly reduced costs. Projects are often less costly than conventional open-cut methods and solve engineering problems such as utility obstacles, poor soils, deep installations and high ground water. Costly lift stations and maintenance costs associated with them are also often eliminated from projects. The societal advantages to this trenchless method include the elimination of traffic delays, road closures, and street repairs as well as the reduction of contaminated soil disposal.

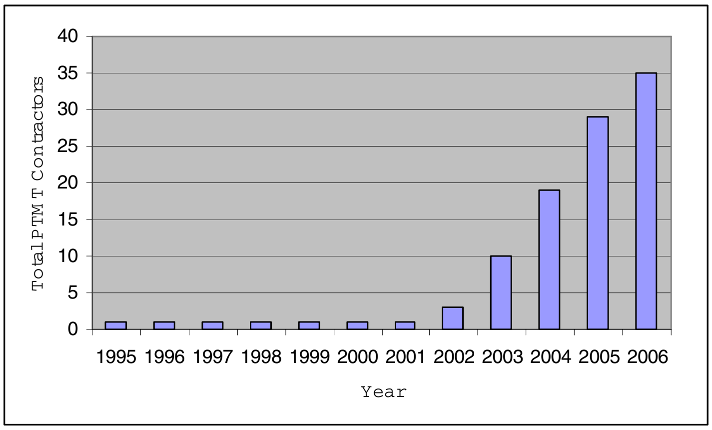

Figure 1- United States PTMT Contractor Growth Chart

Pilot tube microtunneling has been used successfully in weak soils where other methods such as open-cut and auger boring failed. This system works well in weak soils where sewer lines can be installed in zero blow count conditions. Consultants and owners are quite impressed with the rifle-barrel-straight installations that result from this installation method.

2. PTMT- THE PROCESS IN DETAIL

2.1 FIRST STEP

The first step in all the pilot tube installation methods is the precise installation of the

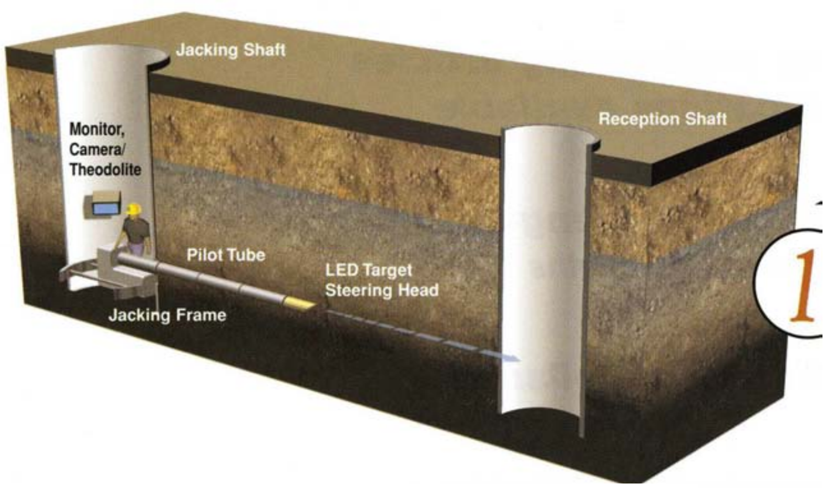

Figure 2- First Step- Pilot tube installation

pilot tube on line and grade. The hollow stem of the pilot tube provides an optical path for the theodolite to display the head position and steering orientation. This step establishes the center line of the new installation as the remaining step(s) will follow this path of the pilot tube. Once step 1 is complete, the theodolite and monitor guidance system may be removed from the jacking pit as they are no longer required.

2.2 GUIDANCE SYSTEM

Video camera mounted above the theodolite transmits the image of the battery powered LED illuminated target located in the steering head to the monitor screen. The straight line indicated on the center of the target designates the direction and path the slant faced steering head will follow.

Figure 3- Theodolite w/ electronic video camera

Figure 4- LED Target

Hollow steel pilot tubes which fasten to each other via threads or clips are available as a double or single wall tube depending on the manufacturer. Only the inner tube

Figure 5- Hollow steel pilot tubes (4 inch diameter)

on the double wall system will rotate with the steering head during advancement. On some systems a bentonite lubricant may be pumped to the steering head to assist with soil friction. These pilot tubes are typically 1 or 2 meters in length as are the casings and product pipe.

A slant faced steering head similar to that of a directional drill houses the LED illuminated target. Steering heads of different degrees of angle are available for the various types of soil. During the installation process the spoil is displaced by the steering head and pilot tube and directed on line and grade by rotation during advancement.

Figure 6- Slant faced steering head

2.3 SECOND STEP

The second step (in the 3 pass and 3 pass modified methods) is to follow the path of the pilot tube with the 3 pass reaming head, which is slightly larger in diameter than

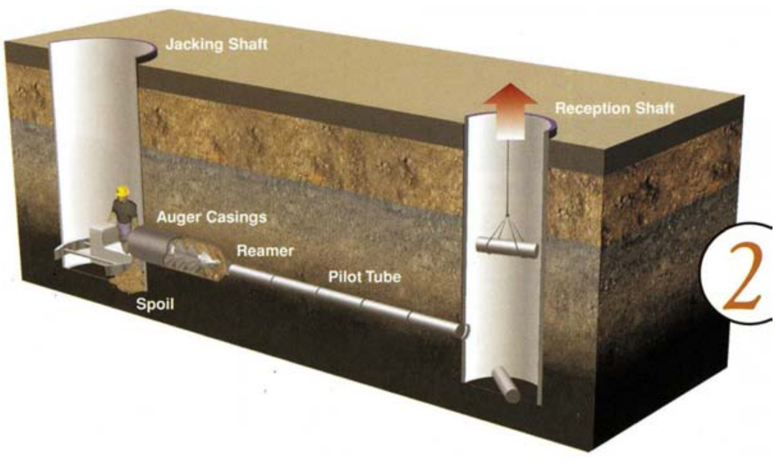

Figure 7- Second Step- Reaming Head/ Auger Casing Installation

the product pipe being installed. The front of the reaming head fastens to the last pilot tube installed in the same manner in which the pilot tubes fasten to each other.

Behind the reaming head follow the auger casings of the same diameter as the head transporting the spoil to the jacking shaft for removal. The spoil can be removed by a muck bucket or vacuum truck depending on the soil type. This step is complete when the reamer and auger casings reach the reception shaft and all spoil is removed.

Figure 8- 1 meter length auger flights and steel casings ready to be lowered and jacked into place for spoil removal on the second step in the three pass method.

Figure 9- 1 meter length by 10 inch ID product pipe which are installed directly behind the auger casings on the third step in a three pass system. The OD of these product pipes are equal to the OD of the previously installed casings.

The second step (the final step in the 2 pass method) is to follow the path of the pilot tube with the 2 pass reaming head which funnels the excavated material into auger casings coupled together inside the product pipe and conveyed through to the jacking shaft for removal. These auger casings are then retracted from the inside of the carrier pipe via the jacking shaft. This method has an advantage to contractors as they are able to install multiple sizes of sewer lines while utilizing the same set of auger casings. The disadvantage to this 2 pass system is the decreased diameter auger casings will limit the maximum diameter of excavatable cobbles and hardened material.

Figure 10- 2 step Reaming Head- Rear View (29 inch OD)

Figure 11- 2 step Reaming Head- Front view (29 inch OD)

The pipes are set into the jacking frame with the auger casings inside. The auger casings are attached to the reamer (if it is the first pipe to be installed) or previous casing followed with the joint make up of the product pipe to the reamer or previous product pipe.

Figure 12- Types of reaming heads.
Top- Rotating cutter head
Middle and Bottom- conventional knife style

Different types of reaming heads are available for a variety of displaceable soil conditions as well as heads capable to control flow when working with as much as 10 to possibly 15 ft below the water table (ultimately depending on the soil type). A swivel is required connecting the pilot tube to the reaming head when a rotating cutter head is used for harder ground.

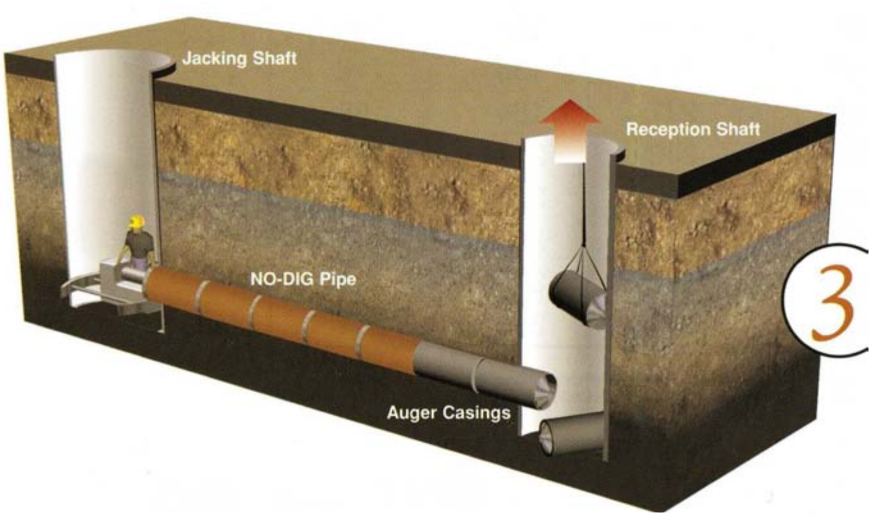

Figure 13- Third Step- Product Pipe Installation

2.4 THIRD STEP

The third step (final step in the 3 pass method) is to install the product pipe once the reaming head reaches the reception shaft. The reaming head and auger casings are pushed into the reception shaft and removed. There is no spoil to be removed in this step as the product pipe has the same outside diameter as the auger casings.

The third step (final step in the 3 pass modified method) is to install a powered cutter head behind the auger casings which is advanced by the product pipe. This method is the newest innovation to the Pilot Tube Methods. The powered cutter head increases the bore to match the larger product pipe diameter. The excavated spoil around the previously installed auger casings is discharged via the reception shaft by reversing the auger direction. This step is complete when the powered cutter head reaches the reception shaft.

Figure 14- Powered Cutter Head

3. APPLICABLE SOILS

The pilot tube system can be used in a variety of soft and displaceable soil conditions. Large cobbles and boulders can cause some challenges during construction. Recent developments such as lubricants for loose sands, water control reaming heads for wet sands, and air hammers for solid rock have increased the possibilities for soil conditions which were once considered impossible. See figure 15 for a list of applicable soil conditions. As with any type of tunneling, a good soils investigation is crucial to the final success of the project.

Figure 15 Applicability of Pilot Tube Microtunneling for Different Soil Conditions

Type of Soil	Applicability
Soft to very soft clays, silt, and organic deposits	Yes
Medium to very stiff clays and silts	Yes
Hard clays and highly weathered shales	Yes
Very loose to loose sands (above water table)	Yes (w/ lubricant)
Medium to dense sands (below the water table)	Yes (to 10 ft. head) Marginal (over 10 ft.)
Medium to dense sands (above the water table)	Yes
Gravels and cobbles less than 2 to 4 in. diameter	Yes
Soils with significant cobbles, boulders, and obstructions larger than 4 to 6 in. diameter	Marginal
Weathered rocks, marls, chalks, and firmly cemented soils	Yes (w/ air hammer)
Significantly weathered to unweathered rocks	Yes (w/ air hammer)

4. VITRIFIED CLAY JACKING PIPE

Vitrified Clay Jacking Pipe has been the predominant pipe material used in the PTMT process due to its high compressive strength (18,000 psi average), low-profile zero-leakage joint, affordability in the typical 1 or 2 meter pipe lengths, and elimination of an external casing pipe. With the guided accuracy of this system there is no need for the typical larger diameter steel casing and the grade-adjusted inner carrier pipe as is necessary with a non-guided boring technique. This saves the additional cost of excavation, transportation, removal of spoil and the purchase of two separate conduits, thus resulting in a lower overall project cost.

The chemical resistance of VCP is unsurpassed, making it the only choice in industrial/commercial applications. The nature of the ceramic material prevents it from changing properties with age, compared to limited life products which experience degradation over time.

Every city in the United States over 100 years old probably has VCP sewer lines in their infrastructure still in service today. These pipelines have lasted, despite having been made with 100 year and older technology and having been installed with outdated construction practices. With today's high tech Vitrified Clay Jacking Pipe and with newer construction practices, engineers are realizing the possibilities for centuries of service life.

5. REFERENCES

Akkerman Inc., Brownsdale, Minn. Available at http://www.akkerman.com/.

BohrTec BmbH, Alsdorf, Germany. Available at http://www.bohrtec.de/.

Herrenknecht, A.G., Schwanau, Germany. Available at http://www.herrenknecht.com/.

Mission Clay Products/ NO-DIG Pipe, Pittsburg, Kansas. Available at http://www.no-dig-pipe.com/.

Najafi, M., and Gokhale, S. (2005). Trenchless Technology: Pipeline and Utility Design, Construction, and Renewal, WEF Press, pp. 283-293.

Wirth GmbH, Division SOLTAU Microtunneling. Available at http://microtunneling.com/soltau/rvs80.htm.

A New Generation of Cementitious Materials for Mortar Lining of Buried Pipes

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Abstract

Concrete (reinforced and non-reinforced) and asbestos cements pipes represent a significant percentage of the storm and wastewater collection systems in many cities across North America. A significant percentage of these pipeline networks are approaching or have already exceeded their 50-year design life. These pipes are in various stages of deterioration, generally due to microbiological induced corrosion or mechanical loading (either external or internal). Open-cut replacement of these pipes, many placed beneath busy streets, is costly and involves a lengthy disruption to normal traffic and business activates. Common rehabilitation techniques for such structures include lining using a thermoplastic or thermosetting liner product or coating using epoxy-based or cementitious-based materials. The latter is in many cases the most economical approach, but the durability of most cementitious lining products in harsh environments limits their useful design life. Hybrid materials, such as polymer concrete, provide superior performance but at higher costs.

Geopolymers are inorganic alumino-silicate polymers that come from chemical reactions under highly alkaline conditions between an active puzzolanic material (such as fly ash or metakaolin) and an activator solution (based on a molar mixture of sodium hydroxide and an alkaline silicate, e.g., sodium or potassium silicate). Geopolymers are usually referred to as inorganic alumino-silicates, and are known for their excellent mechanical properties, high corrosion resistance and chemical stability. Another promising cementitious material is Zeolite, a crystalline puzzolanic aluminosilicate material with uniform molecular sized pores. At present Zeolites are used as catalysts, separation media, ion exchangers and antimicrobial agents. Zeolites can be mixed with sand to form mortar blends that are highly stable in acidic environments.

The paper provides a state-of-the-art review of these two cementitious materials and their chemical and mechanical properties. Thereafter, an experimental testing program undertaken at the Trenchless Technology Center to explore the suitability of these two novel materials for use as coatings for wastewater conveyance, storage and treatment structures is described and its preliminary results reported. Specifically, the effect of various mix design parameters (AS/FA ratio, curing temperature, type of silicate used and $NaSiO_2$ to NaOH ratio) on the compressive strength of geopolymer mortar cubes are reported and discussed.

Geopolymer – An Overview

Among trenchless rehabilitation techniques, Cured in Place (CIPP) is among the most widely used pipe lining method due to its long-term durability and low diameter reduction, however it is often costly for long stretches of large diameter pipes. Slipling is another widely used lining method, but while relatively cost-effective its implementation results in a significant reduction in the pipe cross-sectional area (Sterling, 1999). Epoxy-based coating processes provide excellent protection to the substrate, but the cost of application per unit area is relatively high. The utilization of cementitious materials is in many cases the most economical approach for coating or relining a buried pipe, however durability issues associated with common cementitious lining materials limit their applications and effectiveness in harsh environments. The development of new cementitious materials that could improve the durability characteristics of traditional cementitious coatings could provide a new, technically superior and cost-effective rehabilitation alternative to owners of municipal and industrial pipe networks.

Geopolymers are essentially amorphous inorganic alumino-silicate polymers that form by a chemical reaction under highly alkaline conditions between an active pozzolanic material (such as fly ash or metakaolin) and an activator solution (based on a molar mixture of sodium hydroxide and an alkaline silicate, e.g., sodium or potassium silicate) (Davidovits, 1994). The reaction mechanism of a geopolymer is polymerization rather than hydration (as in the case of hydraulic cements). The polymerization process is carried out by putting the puzzolanic material in contact with the alkaline activator solution, which results in the formation of polymeric chains. It is a temperature aided process, a factor that needs to be considered in outdoor applications in cold climate areas. Hardjito *et al.* (2003) reported that adequate curing temperature for geopolymers range from 30 to 90ºC, depending on the raw materials used and the molar concentrations of the solutions. However, geopolymers are mostly manufactured at temperatures above 60ºC.

Geopolymer cements usually set within a few hours after the beginning of the reaction. The rate of temperature should be controlled to avoid an accelerated loss of moisture which may lead to the propagation of cracks. Geopolymers are somewhat viscous, and the use of super plasticizers is recommended. Geopolymers offer excellent compressive and tensile strengths, superior o that of most rapid-setting Portland cements, and their ultimate strength is reached within three days of hardening (compared with a year or more for OPC).

The chemistry of geopolymer cements is based on silicate aluminates rather than calcium aluminates, and thus these materials are practically inert to sulfate salts attack and possess excellent acid resistance (Hewayde *et al.* 2005; Wallah, 2006). Since it is composed by an alkaline silicate net, these cements are also inert to the alkali-aggregate reaction which commonly happens in Portland cements. In addition, Geopolymers are practically immune to attack by nitric and hydrochloric acid attack (Song, 2005). While exhibiting some degradation during a prolog exposure to sulfuric acid (fly ash typically has around 4-5% of CaO), the silicoaluminate net component of the geopolymer seems to be unaffected, and the material can still retain a great percentage of its structural strength (Allahverdi, 2005). Other attractive characteristics of geopolymer concretes include low permeability and ionic diffusion due to its nano-porous pore system. High fire resistance and very low shrinkage are additional desirable characteristics associated with geopolymer concretes (Gourley and Johnson, 2005). Even though there have been some reports showing that geopolymers may have good adhesion to concrete surfaces (Balaguru, 1998), there are also contradicting research reports. Thus, the adhesion mechanisms between a geopolymer and a concrete surface need to be further researched.

Aside form utilizing a bi-product (fly ash) as a construction material, the substitution of GPC for OPC potentially reduces carbon dioxide emissions by 0.836 t of CO_2 per ton of binder replaced (OPC generates 1 t per ton of binder; GPC generates 0.164 t per ton of binder).

Current commercial applications of geopolymer concretes were reported in Australia, France and the USA. At present time most of the application focus on the production of pre-cast components for chemically harsh environments including pre-cast concrete pipes, railway sleepers and wall panels manufactured by Rocla Technology (Australia). In the USA applications include the production of precast and prestressed concrete elements for the oil industry by Lone Star Industries and Shell Oil Company. In addition to geopolymer concrete, Portland cement-Geopolymer blends were also investigated, with perhaps the most known been PYRAMENT PBC cement, which consists of 80% OPC and 20% GPC. This product is used for the repair of airport runways due to its very high early strength gain (strong enough to drive on in 4 hours).

The Trenchless Technology Center at Louisiana Tech University is working with selected industrial partners on the development of mix designs for the fabrication of pre-cast elements such as pipes and manhole components, in-situ casting of geopolymer concrete for the repair of infrastructure components that were damaged due to contact with corrosive agents, and on spray-on coating of buried pipes and tunnels. Geopolymer specimens are shown in Figure 1.

Fig. 1. Metakaolin-based geopolymer paste (left-hand side); Geopolymer cylinders 30 min following casting; notice smoothness of outer surface (right-hand side)

Zeolites – An Overview

Zeolites are crystalline puzzolanic alumino-silicate materials with uniform molecular sized pores. Zeolites are highly porous, and thus provide ample of room for biological and chemical reactions. At present, they are used mainly as catalysts, separation media, ion exchangers and antimicrobial agents. Zeolite has been used in the cement industries in China, where it used as a 10% replacement to Portland cement in concrete production. It has been found that a high strength concrete with w/c ratio 0.31 to 0.35 and compressive strengths in the order of 80 MPa can be produce in the presence of Zeolite. Zeolite-amended cements exhibit high late strengths and low heats of hydration; however their early strength is low. Furthermore, zeolite-mended cements require more water compared to Portland cements to produce paste of the same consistency. This drawback can be overcome with the use of superplasticizers.

Zeolites contain metals ions, such as calcium and sodium which are easily exchangeable by silver, copper and zinc ions to produce zeolites with antimicrobial activity. It has been demonstrated that zeolites coatings can be highly hydrophilic and antimicrobial. Zeolitic coating with thickness of 4-6 μm was found to have excellent adhesion to various substrates in space applications. Furthermore, Zeolite-based coatings were found to be high resistant when exposed to high acidic and basic solutions (Haile, 2006). For structural applications, zeolites should not be used alone, but in combination with Portland cements. Their highly porous structure produces low mechanical strength, although they improve Portland cement's late strength. The most important property of zeolites is that of being able to exchange metal ions, and therefore be suitable for the immobilization of toxic residues in water and other fluids.

The resistance to corrosion induced by zeolite-amended Portland cement coating might be achieved in one of two ways: 1) using a silver-exchanged zeolite that will provide a bactericide media for the cement coating; or, 2) by lowering the total amount of C_3A in the Portland cement-zeolite admixture. However, the synthesis of zeolites is currently an expensive process and therefore their application for structural purposes is limited.

Research Plan

In 2005 the Trenchless Technology Center at Louisiana Tech University launched a research initiative for developing new coating systems using advanced cementitious materials. The research is currently focusing on the evaluation of geopolymer cement as an alternative cementitious material for the lining of concrete sewer pipes.

The flowing sections describe the preparation of geopolymer cement and the evaluation of the relationship between the main components of geopolymer cement and its compressive strength. The mix design utilized in the pilot testing program included:

- Fly ash from a source in Avon Lake, OH
- Sodium silicate type "N" and "D"
- Sodium hydroxide in pellets
- ASTM-778 Standard sand (1:1)

The experimental design is presented in Table 1. A 14 M NaOH solution was prepared by dissolving NaOH pellets in distilled water and allowing the solution to cool for one day. An activator solution (AS) was prepared by mixing NaOH and $NaSiO_2$ in the desired proportions. The fly ash and sand were dry-mixed in a pan, and the activator solution was added gradually to the mixture while it was stirring. The geopolymer cement mortar was cast in on 5x5x5 cm molds and oven-cured at 60 or 90 degrees for one, two and three days prior to testing. The samples were sealed with plastic sheets to prevent moisture loss. A universal compression testing machine was used to perform the compression tests as per ASTM standards.

Table 1. Design of experiments for the first step of the research plan

RESEARCH VARIABLE	*LEVELS*
Curing time	1, 2 and 3 days
Curing temperature	60 and 90 C
Activator solution/Fly ash ratio (AS/FA)	0.54 and 0.82
$NaSiO_2$/NaOH ratio	1.5 and 2.5

Results and Discussion

Figure 2 depicted the compressive strength (psi) versus the duration of curing for geopolymer mortars made of silicate 'D' (a NaO/SiO_2 ratio of 2) and silicate 'N' (NaO/SiO_2 ratio of 3.5). It was found that silicate 'D' performs better than silicate 'N' in terms of compressive strength regardless of the curing time. Thus, all following tests were performed using silicate 'D'.

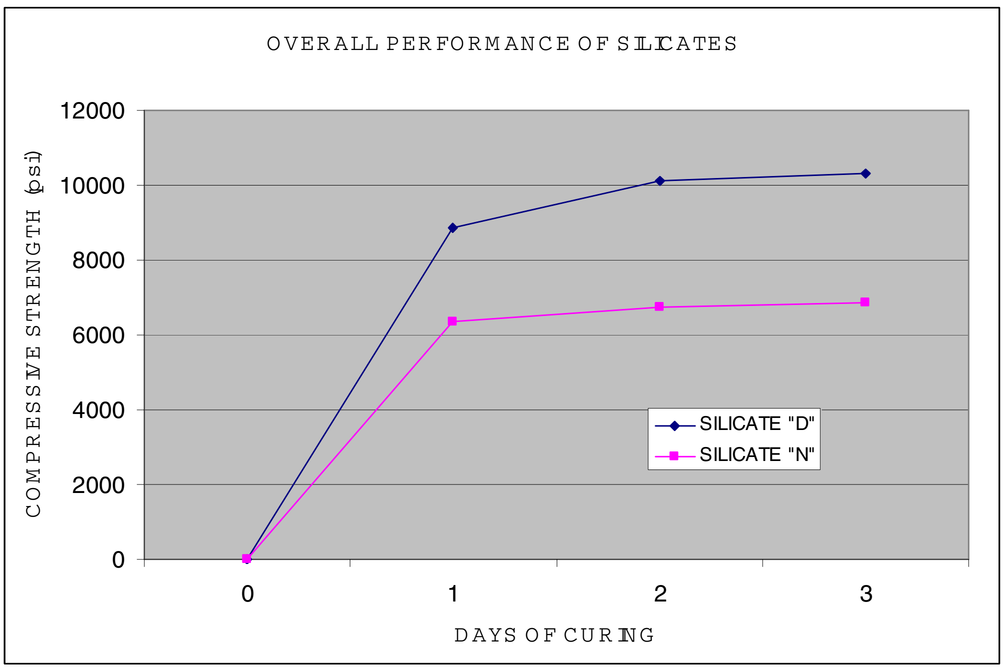

Fig. 2. Type of Silicate vs. Compressive Strength

Figure 3 display the compressive strength vs. curing time. It can be seen that an activator solution to fly ash (AS/FA) ratio of 0.53 gives slightly better results than a ratio of 0.82, with the three-day compressive strength of the mortar approaching 11,000 psi (~75 MPa). This ratio could be further decrease with the aid of super-plasticizers.

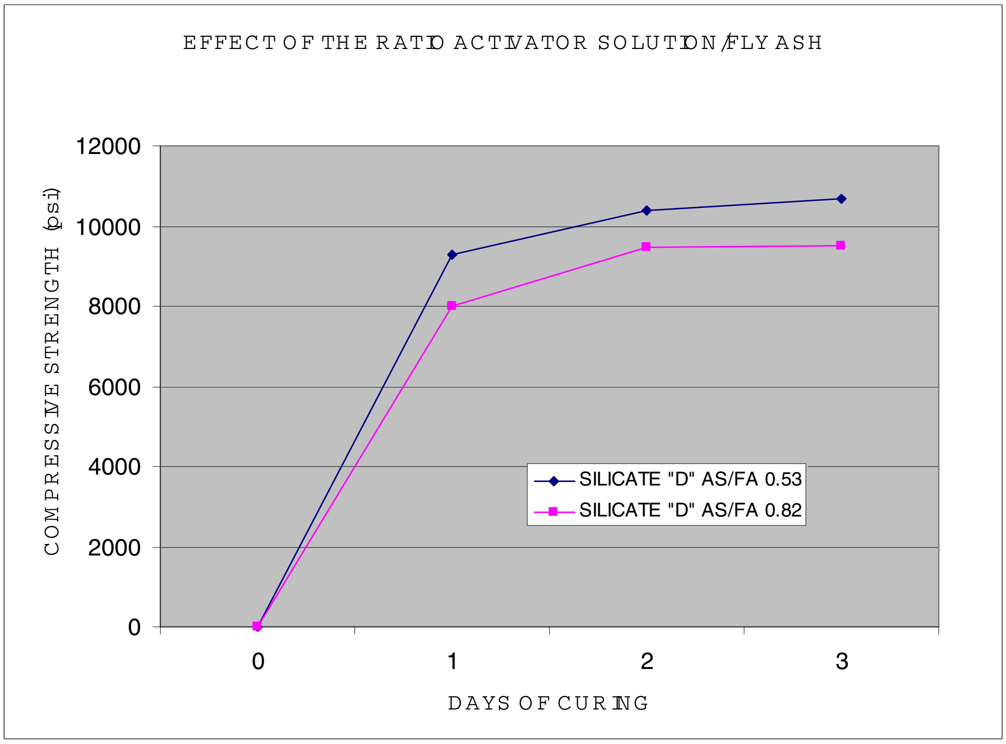

Fig. 3. Effect of Activator Solution to Fly Ash Ratio on Compressive Strength

Figure 3 show that the $NaSiO_2$ to NaOH ratio has little effect on the compressive strength. For cost considerations a ratio of 1.5 was chosen for the next phase.

The effect of curing temperature on compressive strength can be seen in Figure 4. While a curing temperature of 90° C exhibit a slightly better performance after 24 hours of curing, little if any advantage can be observe for longer curing periods when compared to a curing temperature of 60° C. Thus, for economic reason it is recommended to use a curing temperature of 60° C.

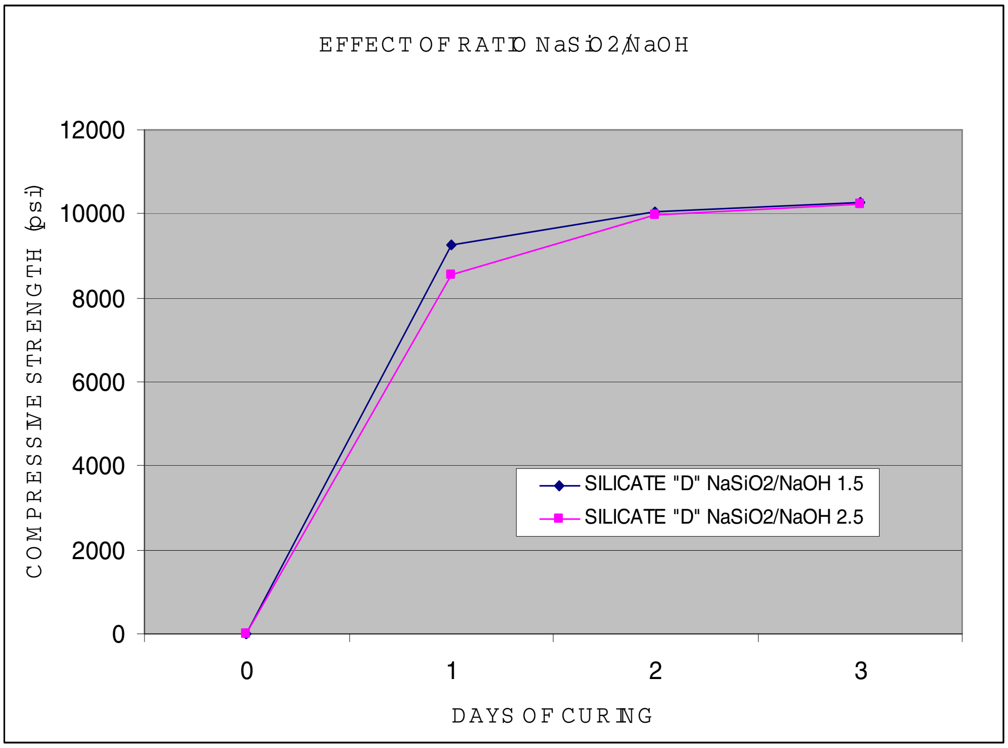

Fig. 3. Effect of $NaSiO_2$ to NaOH Ratio on Compressive Strength

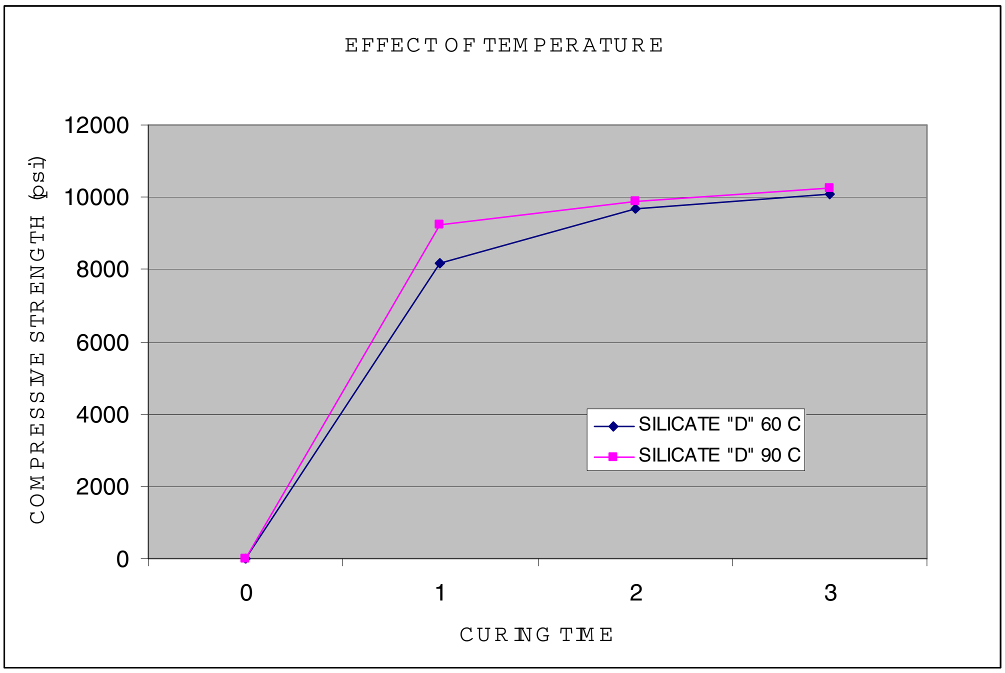

Fig. 4. Effect of Curing Temperature on Compressive Strength

Planned Experimental Schedule

The next phase of the research program include a systematic evaluation of three different commercial sources of fly ash for curing times and chemical resistance, two key parameters for a spray-on pipe coating applications. Data reported in this paper will be used to fix some of the design parameters, namely: Silicate Type 'D' and $NaSiO_2$/NAOH = 1.5. A further reduction in the ratio of activator solution to fly ash will be attempted, as it is likely to improve the compressive strength and reduce the overall cost. Shorter curing times will also be considered for the next step, since they are desirable for pipe lining applications. Intermediate temperatures between room and 90° C will be attempted to optimize the curing time and energy costs. Upon completion of the second phase, additional parameters including viscosity, setting times and adhesion will be studied. The final phase will include a pilot field test during which geopolymer coating will be applied to a real-world sewer pipe.

Summary and Future Work

Geopolymers concrete hold much promise as a new construction material for the conveyance and storage municipal and industrial effluent steams due to their high stability in acidic environments, corrosion-resistance characteristics and mechanical properties including high compression and flexural strengths, resistance to abrasion and wear and low thermal expansion coefficients. The present article reports the results of compressive strength tests of mortar cubes for several formulations of geopolymer cements. The optimal values for several mix design parameters including silicate type and $NaSiO_2$/NaOH ratio were identified. Mortar compressive strengths as high as 95 MPa (14,000 psi) were observed.

Phase II of the research program is currently underway at the TTC, aiming at evaluating the adequacy of several local sources of fly ash for the manufacturing of geopolymer cement suitable for structural applications in harsh acidic environments. Eighteen mix designs will be subjected for a wide rang of mechanical and chemicals tests in controlled conditions in the laboratory. The best performing mix designs will be subjected to pilot-field tests in a several industrial sites in Louisiana known to have aggressive effluent streams.

References

Allahverdi, Ali, *Sulfuric Attack on Hardened Paste of Geopolymer Cements,* (2005), College of Chemical Engineering, Iran University of Science and Technology, Tehran, Iran.

Balaguru, D.P. (1998), "Geopolymer for Protective Coating of Transportation Infrastructures".

Davidovits, J. (1994) "Properties of Geopolymer Cements", Geopolymer Institute.

Hardjito, D., *et al* (2003) "Geopolymer Concrete: Turn Waste Into Environmentally Friendly Concrete", International Conference on Recent Trends in Concrete Technology and Structures, INCONTEST.

Haile T.G., Nakhla, G. (2006) "Protection of Concrete Pipes from Bacterial-induced Corrosion", M.S. Research Project Proposal. University of Western Ontario.

Hewayde, E., Allouche, E.N. and Nakhla, G.F. (2005). “The Use of Metakaolin and Geopolymer Cement to Improve Concrete Resistance to Sulfuric Acid Attack”, Proceedings, 7th International Symposium on Utilization of High-Strength/High Performance Concrete (ACI SP-228), pp. 1453-1466, *ACI, Farmington, MI.*

Sterling, Raymond L., Thorne, James D., *et al*. (1999) "Trenchless Technology Applications in Public Works", p. 22-30.

Song, Xiu-jiang, *Investigation of Cracking Developed in Sulfuric Acid Resistant Concretes*, (2005) ACCI, School of Civil and Environmental Engineering, UNSW, Sydney, Australia.

Wallah, S.E., and Rangan, B.V. (2006), “Low-calcium fly ash geopolymer concrete: long term properties” Faculty of Engineering, Curtin University of Technology.

Use of Nanomaterials for Concrete Pipe Protection

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Abstract

This paper describes an innovative method for mitigating microbial induced corrosion (MIC) in concrete sewer pipes by driving nano-particles containing an antimicrobial agent into the wall of the steel-reinforced cementitious pipe using electrokinetics, an electrical deposition method. Electrokinetic coating involves the application of a weak electric field between the pipe reinforcement and a properly charged solution of nanoparticles. The electrical potential difference drives the nano-particles in the solution into the hardened concrete paste via its pore structure, simultaneously transporting the antimicrobial agent(s) into the pores. This new method of coating creates a mechanical anchorage between the coating liquid and the host matrix, which has inherent advantage over traditional brush or spray-on coating operations. The paper provides a brief overview of recent advances in spray-on coating and lining methods for protecting concrete pipe used in wastewater conveyance systems. Thereafter, a brief description of electrokinetics is provided. Preliminary results of an experimental testing program involving electro-kinetically coating of 35 mock concrete pipe specimens using cuprous oxide, a heavy metal oxide known the have a toxic effect on sulfate reducing bacteria (SRB), are presented. The preliminary testing program focused on examining the effectiveness of using electrokinetics for the treatment of new pipes, partially deteriorated pipes and pipes that exhibited moderate-to-severe deterioration due to sulfuric acid attack. The design of the experiment and the execution of the electrokinetic coating process are also described. The results suggest that the approach could potentially serves as an effective method for coating of new and partially deteriorated reinforced concrete pipes.

Introduction

Microbial induced corrosion (MIC) in concrete sewer conveyance systems is one of the most common types of deterioration encountered in such structures. This corrosion involves a chemical reaction between hydration products in the hardened concrete and biologically produced sulfuric acid, which alter the concrete chemical composition leading to early deterioration, loss of strength and, in extreme cases, pipe collapse due to inability to resist external earth and live loads.

MIC often involves two types of bacteria which are common to many sanitary sewer systems, namely sulfate reducing bacteria (SRB) *Desulfovibrio desulfuricans* and sulfide oxidizing bacteria *Thiobacillus thiooxidans* (also known as 'concretivorous', or 'concrete eater'). Sulfur present in sewage is transformed into hydrogen sulfide by sulfate reducing bacteria which tend to grow on the wet perimeter of the sewer pipe. In areas of turbulent flow, the hydrogen sulfide is released into pipe's head space. The Thiobacillus bacteria, which grow on the sewer crown region above the water line, convert the hydrogen sulfide to sulfuric acid in presence of oxygen (Pomeroy, 1946). The chemical reaction associated with production of sulfuric acid is (EPA, 1992):

$$H_2S_{(g)} + 2O_2 \longrightarrow H_2SO_4 \quad (1)$$

The Corrosion Mechanism

A freshly placed concrete has a pH of approximately 11 – 13, depending upon the mix design. This pH is due to the formation of a stable compound, calcium hydroxide $Ca(OH)_2$, a common by-product produced during the hydration of cement. This initial high pH on the surface of the concrete will not allow any bacterial growth; however, this high pH state lasts for only a short period of time and the pH level slowly declines over time, due to interaction with carbon dioxide (CO_2) and hydrogen sulfide (H_2S) gases. These gases form weak acidic solutions when dissolved in water (carbon dioxide forms carbonic acid and hydrogen sulfide forms thiosulfuric acid and polythionic acid), that lower the pH of the concrete surface to 9 or 9.5. Thiobacillus bacteria that have a unique ability to convert hydrogen sulfide to sulfuric acid in the presence of oxygen start colonizing at a pH of about 9. The first colony of bacteria reduces the surface pH value from 9 to 6.5 by excreting sulfuric acid. These bacteria will not be able to survive at pH values lower than 6.5, and thus the colony dies and the residence is then taken over by another species that is capable of surviving below a pH of 6.5. This newly formed species of Thiobacillus further reduces the pH value from 6.5 to 4 before dying off. This process of colonizing and dying of bacteria continues, until the pH at the surface of the concrete can be brought to a value as low as 1 or 0.5 (Islander et al., 1991). The sulfuric acid produced by the bacteria interacts chemically with the hydration products in the hardened concrete paste. The chemical reactions involved in corrosion of concrete are:

$$H_2SO_4 + CaSi \longleftrightarrow CaSO_4 + 2H^+ \quad (2)$$

$$H_2SO_4 + CaCO_3 \longleftrightarrow CaSO_4 + H_2CO_3 \quad (3)$$

$$H_2SO_4 + Ca(OH)_2 \longleftrightarrow CaSO_4 + 2H_2O \quad (4)$$

The primary product produced during concrete decomposition by sulfuric acid is calcium sulfate ($CaSO_4$), that when it hydrates becomes 'gypsum'. This material has low structural strength, especially when wet. It is usually present in corroded sewers as a pasty white mass at the crown region above the water line. Due to its lower shear strength and poor adhesion, the gypsum is washed off by the wastewater flow, exposing fresh concrete to further acid attack. An example of severe MIC in a 27 year-old concrete sewer pipe is shown in Figure 1.

Figure 1. Close-up of a 36 in concrete sewer pipes that experienced severe MIC. The steel reinforcement ribs are either gone or in an advanced corrosion state. The white solids are MIC corrosion products (marked by red circles)

Current Approaches

A large number of methods have been developed to mitigate MIC in wastewater collection systems. The following discussion will consider only approaches that involve applying a protective coat to the host pipe internal surface. Spray lining is the most widely used coating technique. However this method suffers from several disadvantages. It is often difficult to create proper adhesion between the coating material and the internal surface of the pipe. Also, the "rebound" effect from hurling the coating material at the pipe's wall can result in uneven coating thickness. Combined with a non-uniform application, this uneven layer thickness tends to result in an uneven stress loading and subsequent premature failure. In summary, since the coating material does not penetrate the pipe surface, it tends to flake and fall down with time under loading from shrinkage, mechanical loading from the transported media and/or cracks induced by differential settlement of the pipe (Dietrich, 1989; James, 2003).

Another common method of protection is lining of the pipe internal surface with a chemically stable material. There are different methods of lining such as cured in place pipe (CIPP), fold and formed in place (FFP) and Deform-reform pipe

(DRP), Rotaloc liner and Rib Loc Expanda liner. CIPP employs a chemically stable fabric with a thermosetting resin, where as FFP, DRP, Rotaloc liner and Rib loc expanda liner methods use PVC or HDPE pipe to internally line the deteriorated host pipe. Properly applied, these methods will serve their purpose for a long period of time. The short comings of lining methods include the relatively high costs associated with set up and installation and a close attention to quality control in the remote lining processes. In addition some methods require a by-pass.

Literature Review – Recent Advances in Surface Treatment Methods for Concrete Sewer Pipes

Controlling Sewer Crown Corrosion Using the Crown Spray Process (James, 2003): Originally proposed by Esfandi (1986) and later adopted and further developed by the sanitation group of Los Angles County, California, the "crown spray" process involve a spray system mounted on a floating platform that sprays the crown region of the sewer pipe with magnesium hydroxide. The purpose of crown spray process is to neutralize the acid that has been generated by the Thiobacillus bacteria at the crown region of the pipe by spraying it with a high pH material (magnesium hydroxide), thus elevating the region's alkalinity. The spray process needs to be repeated on the regular basis (time between applications approximately 6-9 months) due to depletion and washout of the magnesium hydroxide.

Protection against MIC using 100% solid polyurethane (Shiwei, 2006): A method of treating concrete pipes against MIC by employing 100% solid polyurethane instead of PVC and coal tar. The 100% solid polyurethane coating was developed at Madison Chemicals, and is claimed to be more economical than PVC and more reliable than coal tar. Test results are provided for pressure, performance and bond strength. The solid polyurethane coating was tested under laboratory and full-scale conditions for a 5-month period. During the tests, no defects such as blistering, cracking, spalling or sticking were noticed. The product was also reported to perform satisfactorily when tested for chemical resistance (vapor and liquid phase tests) and bond strength (ASTM D4541 and ASTM C321).

Corrosion Control in Concrete Pipe and Manholes, (William, 1998): Conshild is an antimicrobial chemical which is applied to the internal wall of the pipe to inhibit the growth of bacteria on the pipe surface. It is typically applied by mixing it with concrete and spraying it as shotcrete. Other application modes include mixing it with an epoxy, with the mixture applied to the concrete surface using spraying equipment.

Effect of metal oxide coating on generation of sulfide in concrete sewer pipes (Hewayde et. al, 2005):
Hewayde et al. studied coating a concrete pipe with copper oxide (CO) and silver oxide (SO) to provide bio-toxic surfaces. The experiment consisted of three concrete pipes, out of which two were internally coated with CO and SO, while the third served as a control. The heavy metal oxides were mixed with epoxy and sprayed on the internal surface of the pipe to a thickness of 350 microns.

Specimens were then filled to a two third depth, with a nutrient solution that has a sulfate concentration of 350 mg/L and dissolved oxygen concentration of 0.5 gm/L. Next, six liters of bacterial solution that had *Desulfovibro Desulfuricans* bacteria were added to each of the tanks. The pipes were then filled to the brim with a nutrient solution. The flow inside the pipe was maintained at 26 L/min, simulating a worse case condition in gravity sewer pipes. At the conclusion of the test period the bacterial growth in the coated pipes was found to be significantly lower (by up to 99%) than that of the control pipe. Also the formation of slime layer was almost absent in the coated pipes. This experiment proved that heavy metal oxides such as CO and SO at low concentrations have a considerable toxic effect on the bacteria species responsible for generation of the hydrogen sulfide and sulfuric acid. A potential problem identified during the study was the leaching of silver oxide into the nutrient solution due to poor bonding between silver oxide and epoxy, which led to SO flaking and mixing with the nutrient solution.

Electrokinetics – An Overview

Electrokinetics is a technique that uses a low DC voltage across the electrodes to move particles or contaminants in a porous medium. Electrokinetics comprises three major processes – electrophoresis, electro-osmosis and electro-migration. Electrophoresis is the movement of colloids, or charged particles, under the influence of an electrical field; electro-osmosis is the movement of fluid, or, in this case, coating solution, through the capillaries and pores from the anode to the cathode. Electro-migration is the movement of ions or ionic complexes.

One of the earliest applications of electrokinetics took place during the 1960's, as part of an attempt to extract heavy metals and organic contaminants from soils and ground water (Acer 1993). Since then it has been used as basis for many other applications including:

1. Removal of heavy metals from mine tailings (Kim et al, 2002).
2. A permeation technique for injecting grouting material into silty soils to enhance their strength properties (Thevanayagam, 2003)
3. Chloride extrication and re-alkalization of reinforced concrete (Velivasakies et al., 1998).

Proposed Work

This paper describes a innovative technique that can be used to coat a concrete pipe using electrokinetics. The process was developed to overcome problems such as poor adhesion, flaking and shrinkage of a coating material that were encountered in traditional spray-on coating processes. The benefits of coating a pipe with heavy metal oxides were demonstrated by Hewayde et al (2005). The present work aims to address the adhesion problem, and focuses on the development of a new coating process in which nano-scale particles of cuprous oxide are driven into the concrete under the influence of a weak electric field.

Experimental Procedure

The experimental design involves a total of 35 mock concrete pipes. The results reported in this paper focus on the testing and results from a pilot study of three specimens, intended to provide an indication of the effectiveness of the proposed coating procedure for new concrete pipes, moderately deteriorated concrete pipes and severely deteriorated concrete pipes. Of the three specimens, one serves as a control (e.g., newly installed pipe), the second specimen was degraded to represent a partly corroded pipe (i.e., exposure of the coarse aggregates), while the third specimen was degraded to represent a moderate-to-severe corroded pipe (loss of coarse aggregates; random exposure of steel reinforcement). The degradation of the specimens was accomplished by filling a cavity in their center with sulfuric acid solution (pH = 0.7), which was refreshed regularly until the desired effect was reached. Following the surface preparation treatment, the three specimens were electrokinetically coated using a cuprous oxide solution. Following the coating process, a series of optical and chemical tests were undertaken to evaluate the effectiveness of this coating process in terms of the extent of penetration of the nanoparticles into the concrete and the concentration of the heavy metal as a function of depth.

Each of the mock concrete pipes was cylindrical in shape, 12″ in height and 6″ in diameter. A void was formed in the center of each cylindrical specimen. The void started at the top surface of the specimen, had a diameter of 1.5″ and extended to a depth of 10″, allowing the specimen to serve as a liquid holding container. The concrete mix design for all specimens consists of Type II Portland cement, water-cement ratio of 0.45, and 0.375″diameter coarse aggregates. A super plasticizer was added to the mix at a ratio of 8 fl oz per 100 lbs of cement to reduce the vibration force required for proper consolidation of the specimen. The total amount of concrete mixed to prepare 35 specimens was 926 lbs, which included 205 lbs of Portland cement, 92.5 lbs of water, 302 lbs of fine aggregate and 326 lbs of coarse aggregate.(ACI 211.1-91)

Commercially available plastic cylindrical molds (12 x 6 in) were used to prepare the specimens. The central void was formed by inserting a 1.5″ diameter PVC pipe into the center of the mold. The lower end of the PVC pipe was blocked using expandable foam to prevent the fresh concrete from entering the cavity. The PVC insert was secured to the mold to prevent it from ‘floating’ during the casting of the concrete.

The steel reinforcement consists of fine gauge welded wire reinforcement fabric that served as both transverse and longitudinal reinforcement. The mesh was cut into rectangular shapes of dimensions 11.5 in x 12.5 in. The reinforcement fabric was then wrapped around four 0.25″ diameter threaded rods to form a cylinder 4″ in diameter. Steel ties were used to secure the reinforcement fabric and threaded rods. The threaded rods were 14″ in length and served both as a longitudinal reinforcement as well as the terminals for the power supply. Figure 2 presents a schematic diagram of a mock pipe specimen.

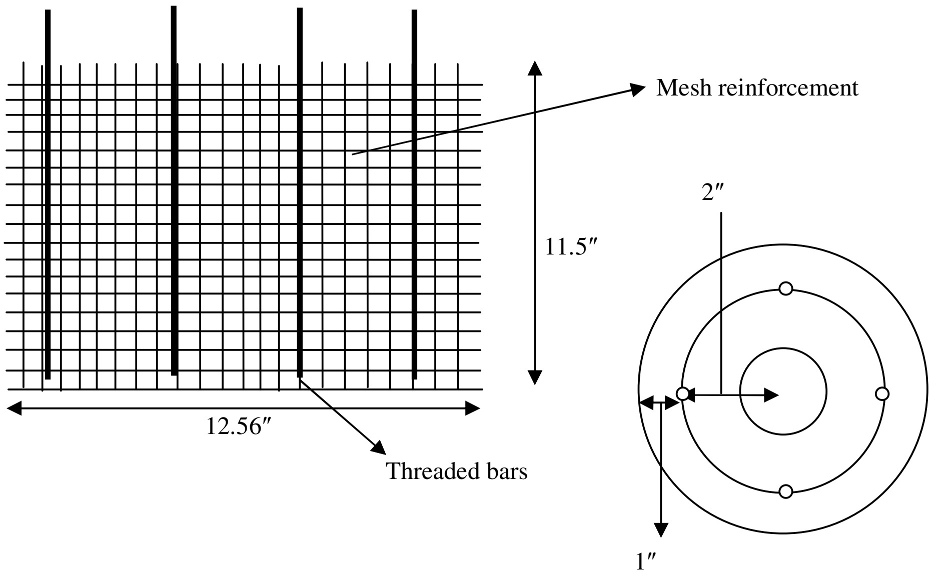


Figure 2. Schematic diagram of a mock pipe specimen

The reinforcement cage was placed inside the cylindrical mold and wet concrete was poured, while the PVC insert was maintained at the center of the mold. The PVC insert was removed three hours following placement of the concrete with care so as not to disturb the concrete specimen. The specimens were allowed to cure in a water holding tank for a period of 28 days (as per ASTM C 31). Following the curing period the specimens were removed from the tank and air dried. No breaking agents were applied to the PVC pipe inserts to avoid altering the surface characteristic of the specimens' inner wall. The 28 days compressive strength of the batch was obtain by conducting a standard compression test on 3 solid cylinder specimens, resulting in an average compressive strength of 5456 psi.

Solution Preparation

The chemical solution used for cuprous oxide deposition was prepared based partially on work reported by Jongh et al (1999), which successfully electrodeposited Cu_2O (cuprous oxide) on a transparent conducting substrate using an alkaline Cu (II) lactate solution to keep the cuprous oxide in suspension. The pH of the solution was maintained between 8 and 9 to get a maximum amount of deposition. Forty five gm of Cu_2SO_4 (copper sulfate) were dissolved in 75 mL of lactic acid to obtain the copper lactate solution. Next, 225 mL of 5 M NaOH (sodium hydroxide) was added in a titration-like fashion to maintain the pH of the solution. The final product was the dark blue colored solution shown in Figure 3.

The solution was stirred for 12 hrs prior to being poured into the void in the concrete specimens. A copper rod was placed in the solution to serve as the anode, while the steel reinforcement in the concrete served as the cathode. The electrokinetics process took place by applying 0.3 Amp of current continuously over a period of 5 hrs. The time of application of the electrical current was selected as the maximum beneficial period for such an electrokinetics process (i.e., optimal conditions). It is expected that in practical applications this time period will be greatly reduced while maintaining bulk of the benefits for the electrokinetics process.

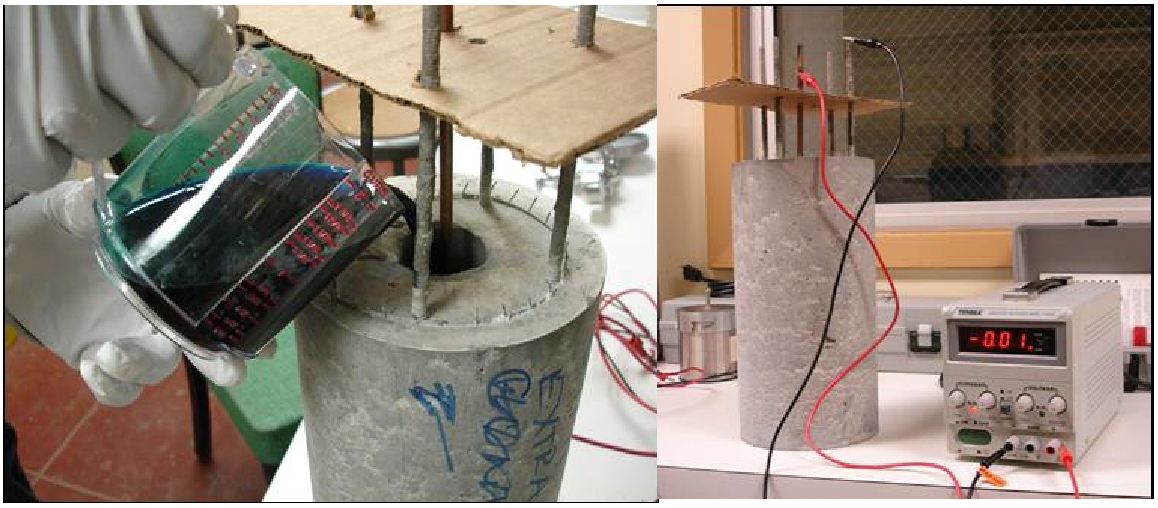
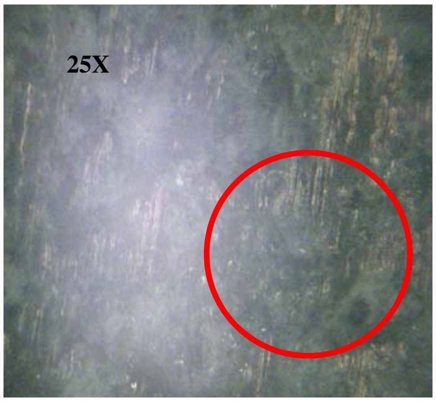


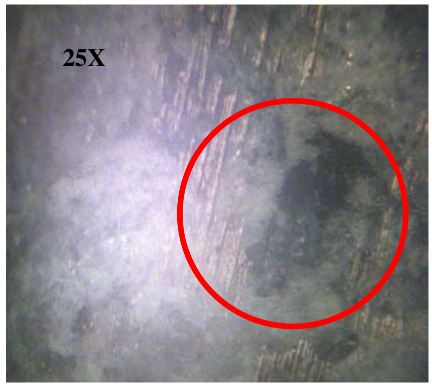
Figure 3. cuprous oxide lactate solution poured into the a specimen (left); The electrokinetic coating process underway, note the copper electrode at the center of void (right)

Preliminary Results

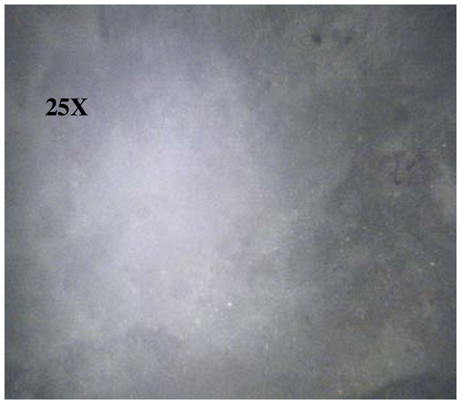
Unaltered Specimen (control; fresh concrete pipe): Following the electrokinetics coating process, the solution remaining in each of the specimen was removed and the inner wall of the "pipe" exposed by cutting 'slices' using a masonry diamond saw. The slices were inspected using a powerful optical microscope for evidence of copper particles deposited in the concrete matrix. The images from the optical microscopy examination revealed what appeared to be copper oxide particles deposition inside the concrete matrix. Figure 4 presents selected snap shots from the microscopy examination. Copper depositions can be identified as green color areas on the gray-white background of the hardened cement matrix. Images 4a and 4b display distinct green areas within the gray-white background typical for hardened Portland cement paste. These areas are suspected to be traces of copper oxide driven into the concrete matrix via the electrokinetics process. Images 4a and 4b were taken from the non-corroded and partially corroded specimens, respectively. In contrast, image 4c displays a uniform gray color, with no traces of green areas. This suggests that the copper-oxide did not penetrate into this specimen during the electrokinetics process. A possible explanation is that the weak electrical field is present between the anode and cathode only. Exposing the reinforcement resulted in a "short-circuit" with the electrical current gradient flowing primarily to the partially exposed reinforcement rather than moving through the less conductive concrete material.



(a)



(b)



(c)

Figure 4. Images (a) and (b) showing copper-colored traces in the hardened concrete paste in the vicinity of the reinforcement in the new and partially corroded specimens, respectively; image (c) taken from the fully corroded specimen shows no traces of copper oxide. (All images taken using x25 optical microscope).

To confirm the hypothesis that copper oxide particles migrated into the concrete matrix, Atomic Absorption Spectroscopy analysis was performed on the three specimens, (i.e., non-corroded, partially corroded and fully corroded). The results of the AAS analysis are summarized in Table 1.

Table 1. Results of Atomic Absorption Spectroscopy

SAMPLE (pre-treatment)	ABSORBANCE	PPM	% COPPER IN CONCRETE
Non-corroded	0.012	1.231	0.022
Partially corroded	0.015	1.352	0.064
Fully corroded	0.003	0.312	0.009

Discussion and Conclusion

The objective of this paper was to report on the study of a potential new coating process for pre-cast concrete pipes (dry-cast mix design; low w/c ratio; high compaction effort) using electrokinetics. The results demonstrated the feasibility of depositing copper oxide particles inside a dry-cast concrete matrix using a weak electric field. Optical microscopy images appeared to reveal the presence of copper oxide inside the concrete matrix in the case of new and partially deteriorated concrete pipe cylinders treated using the proposed coating process. No penetration of copper-oxide was detected in the fully deteriorated specimen, potentially due to the development of a "short-circuit" via the exposed reinforcement mesh that came into contact with the copper lactate solution. Atomic absorption spectroscopy analysis proved that copper was deposited in the concrete matrix of all 3 specimens. However, in the case of the non-corroded and partially corroded specimens the percentage of copper was considerably higher than in the fully-corroded specimens. The method of treating was shown to be most successful for partially corroded concrete than the non-corroded specimen that had added wall thickness and higher electrical resistance. Possible explanations are the larger surface area due to the corrosion process and the reduced wall thickness resulted in a higher concentration per unit volume between the pipe surface and the reinforcement.

When lactic acid is mixed with copper sulfate, copper lactate and sulfuric acid are formed in the solution phase. With the addition of sodium hydroxide, the solution contained copper, sodium, lactate, sulfate and hydrogen ions. Due to their higher reducing ability, copper ions move towards the cathode to take up electrons and form stable copper (Cu^{+2} to Cu). The formation of cuprous oxide was expected at or near the concrete surface via reduction of the copper ions in presence of oxygen. However, preliminary results indicate that such reduction might not take place to the desired extent, potentially due to lack of free oxygen within the concrete media. As a result the coated surface consists primarily of copper, which is a less effective biocide agent compared with cuprous oxide. The next phase of the study will experiment with various methods of delivering oxygen to the surface of the concrete shortly prior to or during the treatment process, to ensure the formation of cuprous oxide.

Acknowledgment
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References

Acar, Y.B. and Alshawabkeh, A.N. 1993. 'Principles of electrokinetic remediation', *Envir. Sci. and Technol.*, 27(13), 2638-2647.

EPA, 1992, "Detection, Control and Corrosion of Hydrogen Sulfide Corrosion", EPA/1992-312-014/40169.

Hewayde, E., Nakhla, G.F and Allouche, E.N. (2005). 'Beneficial Impact of Coatings on Biological Generation of Sulfides in Concrete Sewer Pipes', *Journal of Structures and Infrastructure Engineering,* Taylor and Francis (in press).

Islander, R.L., Devinny, J.S., Mansfeld, F., Postyn, A., Shih, H, 1991. 'Microbial Ecology of Crown Corrosion in Sewers'. *J. Environ. Engg*, Vol., 117, No.6, pp 751-770.

James K.M., 1970. 'In place Treatment of Foundation Soils', *J. of the Soil Mechanics and Foundations Division*, 96(1), 73-110.

James, J. 2003. 'Controlling Sewer Crown Corrosion Using The Crown Spray Process with Magnesium Hydroxide', Underground Construction Technology Conference, Houston, TX.

Kim, S., Kyoung, W. and Stuben, D. 2002. 'Evaluation of Electrokinetic Removal of Heavy Metals from Tailing Soils', *J. of Environ. Engrg*, 128(8), 705-715.

Parker, C. D., 1946, "The Isolation of Species of Bacterium Associated with the Corrosion of Concrete Exposed to Atmospheres Containing Hydrogen Sulfide", Australian Journal of Experimental Biology and Medical Science, Vol. 23, pp. 81-90.

Jongh, P.E., Vanmaekelbergh, D., and Kelly J., 1999. 'Cu_2O: Electrodeposition and Characterization', *J. Chem. Mater*, 11, 3512- 3517.

Shiwei, G., 2006. Synergistic Protection against Microbiologically Induced Corrosion Using a 100% Solid Polyurethane Incorporated with Antimicrobial agents",

http://www.madisonchemical.com/pdf_tech_papers/Synergistic_Protection_Against_MIC.pdf

Thevanayagam S. and Jia., W. 2003. 'Electro-osmotic Grouting for Liquefaction Mitigation in Silty Soils', 3rd International Specialty Conference on Grouting and Ground Treatment, February 10–12, New Orleans, Louisiana, USA, pp. 1507-1517

William, E. Shook, L. and Bell, W., 1998. 'Corrosion Control in Concrete Pipe and Manholes'**,** Water Environmental Federation, Orlando, Florida. http://www.conshield.com/corrosionControl.htm

Velivasakis, E., Henriksen, S/K. and Whitmore, D., 1998. 'Chloride Extraction and Re-alkalization of Reinforced Concrete to Stop Corrosion", *J. of Performance of Constructed Facilities*, 12(2), 77-84.

Esfandi, E. (1986). "Proposal to Conduct Research for Control of Sewer Corrosion", Internal Memorandum, County Sanitaion Districts of Los Angles County.

Dietrich Stein, (1989). "Rehabilitation and Maintenance of Drains and Sewers", John Wiley & Sons Inc. ISBN: 3-433-01316-0.

Pneumatic Piercing Tools for Last Mile Installations

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ABSTRACT

Piercing tools are being successfully used everyday, in almost every area of utility construction including the most recent Fiber-To-The-Home (FTTH) or Fiber-To-The-Premise (FTTP) phenomena. Today's piercing tools are reliable, accurate and hard working, which makes them particularly well suited for last mile operations.

PNEUMATIC PIERCING TOOL FTTH APPLICATION

With pneumatic piercing tools, small crews can perform accurate bores from 50 to 150 feet in length. Piercing tools can be easily configured to pull in conduit for fiber optic cable installation. Advancements in design have made today's piercing tools more accurate and more powerful than ever before. Minimal operating space is required and the tool serves as a complement to and in many situations an economical alternative to larger, more expensive directional drilling equipment. As Cities and other utilities become more aware of these tools and their capabilities, cross bores have become a large issue for all stake holders. Training of operators, utility locaters, has become critical.

Several large telephone and multi-media corporations are in the midst of installing thousands of miles fiber optic cable to carry television, internet and telephone services. A large portion of this work is dedicated to last mile installation in residential neighborhoods.

By utilizing pneumatic piercing tools, companies such as Verizon are able to avoid considerable lengths of trench work and reduce restoration costs dramatically. Small entry and exit pits are much less disruptive and costly than long deep trenches. Positive social factors are an added benefit of trenchless replacement and installation methods. Numerous job specific case studies support and document the capability, growth, and potential of pneumatic piercing tools in the FTTH industry.

PIERCING TOOL BACKGROUND

The piercing tool is basically a piston within a casing. Compressed air moves the piston and the impact of the piston drives the tool forward. That was the design of the first tool and it is essentially the same in principal today. The earliest tools, or moles as they are often referred to, were often difficult to handle and hard to restart after stopping. Accuracy was also a problem. Because accuracy was such a problem with most piercing tools, the technology was often thought of as unreliable and was not allowed to reach its full potential. The development of the reciprocating stepped-cone chisel-head assembly changed that.

Design improvements improved accuracy. With the introduction of the chisel-head assembly on the certain piercing tool models in the early 1970s, accuracy was no longer an issue. See Figures 1 & 2. The assembly is spring-loaded and pushes forward from the main casing creating a pilot bore for the tool and helping it stay on target. The chisel action and stepped-cone design allow the tool to power through difficult soils and obstructions without being pushed off course."

Figure 1 Note the chisel tip, the stepped cone and the joint where the reciprocating action of the boring tool head takes place. These parts are designed to be replaceable when significant wear has taken place.

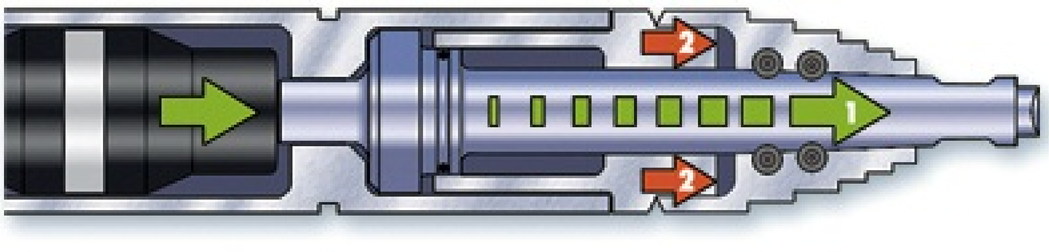

Figure 2
In cross section, the reciprocating action of the head assembly is shown as the piston strikes the chisel, thus driving the head forward each time the piston strikes.

Typically piercing tools range in size from as small as 1 3/4 inches in diameter up to 7 inches and can bore accurately up 150 feet in length. A minimal crew compliment is needed to operate a piercing tool and only small entry and exits pits are required for most projects. The tools can usually be fitted with different types of cones or heads for various soil conditions. In addition, piercing tools can be used to perform a standard bore or they can be outfitted with a range of pipe and cable pulling adapters/accessories to pull in product pipe, conduit or cable while boring.

In addition to horizontal boring, the tools can be used for other applications like pipe bursting and pipe ramming. Because the piercing tool can be used in so many ways, it represents one of the most versatile construction tools available. With some creativity and applied knowledge, contractors and utilities can get an incredible amount of production from these simple tools.

PROJECT BACKGROUND

Verizon is one of the major telecommunication companies pushing fiber-to-the-premises (FTTP) services. The company began implementing its FTTP program in early 2002. Since then the program continues to expand and now includes projects in several states and large cities like Tampa, FL. In May of 2006, Verizon announced that an initial group of more than 40,000 Tampa area households were now able to receive its product offerings through its all-digital fiber optic network. The company had already deployed over 3 million feet of fiber in the Tampa area. By the time the program is scheduled to be complete, five years from now, over 9 million feet of fiber optic cable will be installed.

For a majority of the path work, Arrow crews use 2-inch through 3.75-inch diameter pneumatic piercing tools. See Figure 3. Typically the piercing tool pulls in mule tape. Mule Tape is a form of high strength cloth tape that is used for pulling products through ducts and conduits. It seems to work much better than rope or cable. Once the run is complete, crews pull the conduit in with the mule tape. In certain circumstances they will complete the bore, then attach the conduit to the front of the tool and back the tool through the boring, pulling in the conduit. Depending on soil conditions boring times range from a few minutes to a half an hour.

Figure 3 This illustration shows a well-used piercing tool, note the wear on the chisel tip and stepped cone. These can be replaced at the discretion of the user.

The piercing tool plays a major role in operations. For the Verizon project piercing tools are used daily. On high production days, crews install over 1,000 feet of conduit. Arrow has two 12-man crews boring under driveways and yards everyday. Basically crews are installing single, double and triple conduit runs of 1 ¼" polyethylene. They probably have over a dozen piercing tools and they're working them constantly. Piercing tools are relatively low cost to operate and to purchase. Thus it is not unusual to see numerous Piercing tools operating on a project site at the same time. See Figure 4.

Figure 4 A typical piercing tool setup includes the boring tool, reinforced air hose and lubricator. Compressed air supply is provided by a typical construction air compressor.

Arrow crews are installing conduit for bores and path. The path is the main line that feeds neighborhood. Crews are performing what's called stitch boring with the piercing tool. Crews dig small pits on either side of the driveway. And launch the piercing from one side to the other, then bore under the yard to the next driveway pit. The shots are usually 30 to 40 feet long. Small pits eliminate a large portion of restoration.
See Figures 5 & 6.

Figure 5 A typical excavation for an FTTH service. This excavation is approximately 2 x 4 feet. Depth is 24-30''. Note the reel of conduit, utility boxes, roll of mule tape, turf to be replaced, hand tools and excavated material. Note that the excavated material is placed on plastic sheeting to protect the turf underneath. The mail box was not disturbed.

Figure 6 Launch excavations for Piercing tools are normally quite narrow and just long enough for the tool to lie flat on line and grade of the bore.

In addition to productivity, minimal disruption is a key benefit of the trenchless piercing tool. Restoration is a time consuming and expensive process. It can make up almost 80% of the cost of an installation project. With the cost of restoration so high, trenchless options are extremely attractive, piercing tools especially. Plus the trenchless application is ideal for last mile operations. Arrow Construction crews are working in established neighborhoods with driveways, sidewalks and landscaping. See Figure 7.

Figure 7
Note the final restoration of this residential street. Minimal disruption allows for reuse of the turf that was removed for the bore pits. This is the same location shown in Figure 5.

The Arrow crews, however, are not installing the fiber themselves. They are simply installing the conduit that protects the fiber. Greenberg said the installation of the fiber usually occurs within a week, after the conduit has been completed and inspected. By that time, Arrow Construction crews have moved on to begin boring in the next neighborhood.

A GIS BASED SIMULATION OF GROUND MOVEMENT DUE TO PIPE BURSTING OPERATION

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Abstract

Accurate prediction of the magnitude and extent of ground movements caused by the use of pipe bursting is important if confidence in its use is to be enhanced. This paper presents a method for quantifying the three-dimensional surface displacements induced by pipe bursting, which was implemented using commercial GIS software. Most current analytical solutions for determining soil displacement are based on the plane or controlled strain approaches using the cavity expansion theory, and simulation results are presented in 2-D. In the methodology presented in this paper the ground surface is divided into a grid with a pre-determined mesh size and the coordinates of each node are represented in a spatial data format. A simplified fluid flow theory is used to manipulate the spatial data. As the bursting head advances, the vertical displacement of each node is computed. The profile of the ground surface at each time step (i.e., incremental advance of the bursting head) can be visualized and the predicted soil displacement contour at different elevations studied. The paper presents the model formulation, codification using ArcView GIS software and the results of a validation process, where model's predictions were compared with experiments results reported in the literature.

The incorporation of an analytical geotechnical ground movement algorithm within ArcView provides a powerful analysis tool for utility and municipal engineers. The designer can now export a shape file with existing utility lines and other surface improvements data into the analysis module, run the analysis and obtain the predicted soil movement at various locations within the proposed alignment. Using ArcView's powerful spatial analysis capabilities, potential problem areas (i.e., displacement exceeding a pre-determined threshold in the vicinity of a buried structural element) can be then identified with ease.

Introduction

Underground infrastructure includes water and sewer systems, gas and petroleum pipelines as well as cable utilities such as power and telecommunication. Because many of these systems are located in congested urban areas, installation of a new pipe or rehabilitation of an existing one using conventional open trenching might be disruptive to the surrounding built environment as well as pedestrian and vehicular traffic flow patterns. To eliminate many of the drawbacks associated with these traditional methods, new approaches for buried infrastructure construction methods have evolved that aimed at minimizing disruption by reducing or eliminating excavation requirements during the installation, rehabilitation or replacement of buried services.

Pipe bursting is a trenchless method used to install a new continuous pipe by fragmenting the existing pipe into the surrounding soil, thus allowing the advancement of a new pipe into the newly created cavity. There is a growing demand for the use of the pipe bursting method in North America, particularly for the replacement of cast-iron water main in high and medium density urban areas. The size of the potential pipe replacement market is expected to increase in the future due to the growing need for infrastructure to support new development, deterioration of existing pipe networks, and technological advancements that increase the cost-effectiveness of this technology.

One of the main obstacles for wider acceptance of pipe bursting as a main stream construction method is the potential for ground movement and the resulting damage to surfaces improvements, adjacent foundations and buried services. Pipe bursting involves the generation of radial force to fracture the host pipe and force the fragments into the ground. Thus, potential damage to adjacent services and structures when using this technique is of prime concern.

Literature Review

Chapman *et al.* (1996) developed a simple analytical model based on a modified fluid flow theory to predict soil displacements occurring around pipe bursting operations. The results showed that the fluid flow model was best suited for deeper situations where the increase in vertical stress causes the expansion to become more uniform.

Atalah *et al.* (1998) conducted field and analytical studies of ground movements due to pipe-bursting operations. They reported that ground displacements were dependent on the degree of upsizing, the type and compaction level of the existing soil around the host pipe, and the nature of confinement of the soil around the pipe.

Saber *et al.* (2003) used three-dimensional finite-difference simulation program (FLAC3D) to simulate the cavity expansion within the soil caused by the pipe bursting process and to study soil displacements in the vicinity of the burst pipe and at the ground surface. The results indicated that the soil movement was concentrated in a wedge-shaped zone between the centerline of the old pipe and the soil surface. The magnitude of ground movement was found to be greatest in the vicinity of the old pipe, and decreased in magnitude toward the ground surface.

Lapos (2004) reported ground movement results obtained from a pipe bursting experiment. For the specific conditions tested, the maximum surface heave was measured when the burst head was located directly beneath that point. The ground heave was reduced to a certain residual value once the burst head passed beyond a certain point.

The literature review indicated that the majority of the analytical approaches used to date to determine the displacement of soil have been based on plane or controlled strain approaches using a cavity expansion theory. All the simulation results were presented in two dimensions. Also, no detailed research work has been done on taking into account the contraction case (i.e. the outer diameter of the replacement pipe is smaller than that of the bursting head).

Analytical Model and Methodology Description

Sagaseta (1987) presented a close form solution for obtaining displacement at a point $P(x, y, z)$ caused by a sink or a source at a point $C(x_0, y_0, z_0)$ (Fig.1).

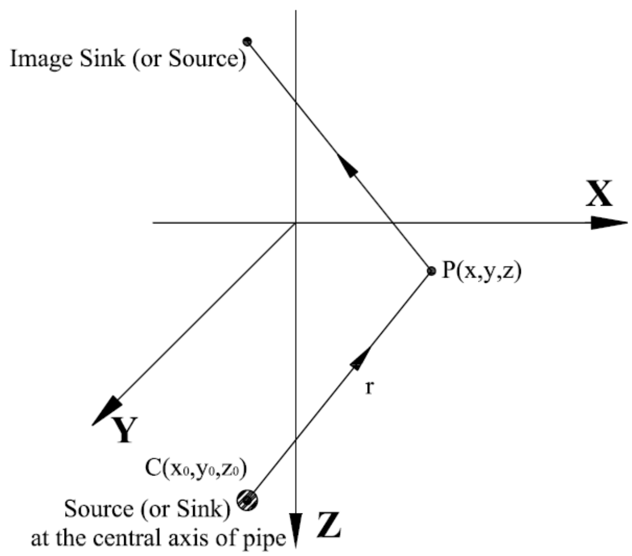

Fig.1 Application of Segaseta's model to pipe bursting operation

The general formula to calculate the radial displacement around a cavity is given by the following equation:

[1] $$S_r(r) = K \cdot \frac{a}{n} \cdot \left(\frac{a}{r}\right)^{\alpha}$$

where K is a constant, a is the radius of the equivalent sphere or cylinder, $n = 2$ in plane stain and $n = 3$ in three dimensions, r is the radial distance from the point sink or point source, $r = \sqrt{(x - x_0)^2 + (y - y_0)^2 + (z - z_0)^2}$. For a sink in dense (dilatant) soil or a source in loose (contracting) soil:

[2] $$\alpha = (n - 1) / \alpha_a$$

For a sink in loose soil or a source in dense soil:

[3] $$\alpha = (n - 1) \cdot \alpha_a$$

Where, $\alpha_a = \dfrac{1 - \sin \nu}{1 + \sin \nu}$ and ν is the angle of dilatancy of the soil.

Sagaseta concluded that the surface movements can be directly obtained simply by multiplying the movements due to the sink (or source) in an infinite medium by a factor of 2.

During pipe bursting operation, the soil around the host pipe and the bursting head are forced to move outwards as the bursting head advances through the host pipe, which is considered to act as a source. If the outer diameter of the replacement pipe is smaller than that of the bursting head, soil will moves inwards to fill the cavity between the fragments of the host pipe and the replacement pipe, which is considered to act as a sink. In order to simplify the simulation, both the source and the sink are assumed to occur at the pipe axis. The ground surface is introduced by using an equal and opposite virtual image of the source or sink (Figure 1).

The simulation can be divided into two phases. In the first phase the bursting head fragments the host pipe into the surrounding soil and then move forward. Soil expansion will lead to the formation of a heave on the ground surface. In the second phase, the replacement pipe enters the newly created cavity. If the outer diameter of the replacement pipe is smaller than that of the bursting head, soil contraction occurs. This results in soil settlement on the ground surface. Thus, ground movement is caused by both, soil expansion and soil contraction.

In the simulation, the ground surface is divided into grids with a pre-determined mesh size. In the examples presented herein it is assumed that the ground surface is flat prior to commencing the pipe bursting operation. However, any surface profile can be utilized in the analysis. The coordinates of each node are in a spatial data format. The model input data include the soil's mechanical properties, pipe's geometry and installation parameters. As the bursting head advances, the vertical displacement of each node is computed for each time step. Thus, the profile of the ground surface at each time step (i.e., incremental advancement of the bursting head) can be visualized, and the predicted soil displacements studied. All data are saved as database file format and then exported into Arcview.

ArcView software powerful capabilities in terms of visualization, managing and analyzing special data, makes it a suitable tool for analyzing and presenting the outcome of spatial data analysis associated with geotechnical problems. Thus, a 3-D visualization of the ground surface for different locations of the bursting head can be accomplished. Figure 2 shows the approach taken for simulating soil displacement at the ground surface. For each time step at each of the nodes, soil displacement equals the sum of its two components, namely soil expansion and soil contraction (if applicable).

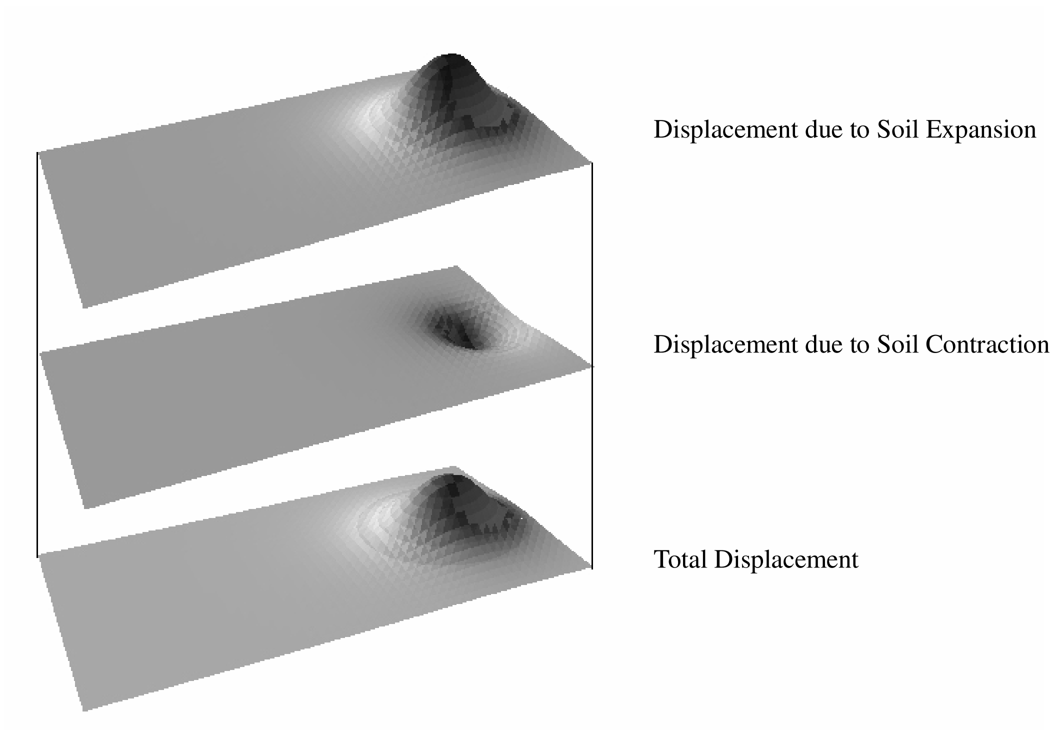

Fig.2 3-D visualization vertical displacement at ground surface

Figure 3 shows the results of a numerical simulation as a function of time for a pipe bursting test conducted in a soil-structure interaction chamber (1.2 m high×4 m long×2 m wide) at Louisiana Tech University (Baumert *et al.*, 2006). A hump developed at the first time step and moves forwards as the bursting head advances. It can be seen that the vertical ground movement decreases gradually as one get further away from the centerline of the host (i.e., existing) pipe. The vertical ground movement increases as the distance to the bursting head decrease. In this specific test (loose sand) the hump settles into a depression shortly after the passage of the bursting head.

Fig. 3(a) Busting head at 0.12 m

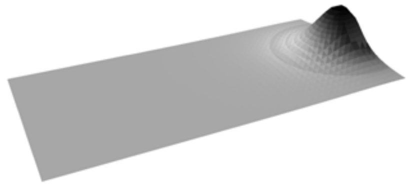

Fig. 3(b) Bursting head at 0.3 m

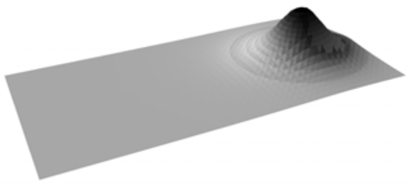

Fig. 3(c) Bursting head at 0.75 m

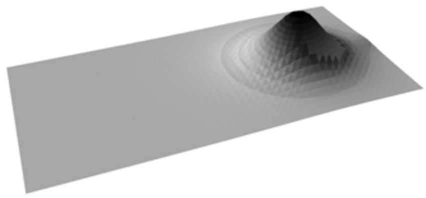

Fig.3 (d) Bursting head at 1.1 m

It can be seen that the surface heave drop quickly as one move away from the centerline of the host pipe. It can also be seen that the ground surface settle following the passage of the bursting head.

Case Study

The analytical model was used to simulate a laboratory pipe bursting test reported by Lapos (2004). The tests were carried out in a 2.0 m wide by 2.0 m long by 1.6 m deep soil box. Some test parameters are given in Table 1.

Table 1. Test Parameters

Depth of cover (mm)	*M.D. of the bursting head (mm)*	*O.D. of the host pipe (mm)*	*O.D. of the replacement pipe (mm)*	*Internal friction angle (^0C)*
685	202	114	159	44

* M.D.– maximum diameter, O.D. – outer diameter

In the simulation, the ground surface is divided into a grid, with a cell size 50 mm by 50 mm, to provide a 3-D visualization of the ground surface profile using ArcView. A schematic drawing of the idealization of the experimental setup is given n Figure 4.

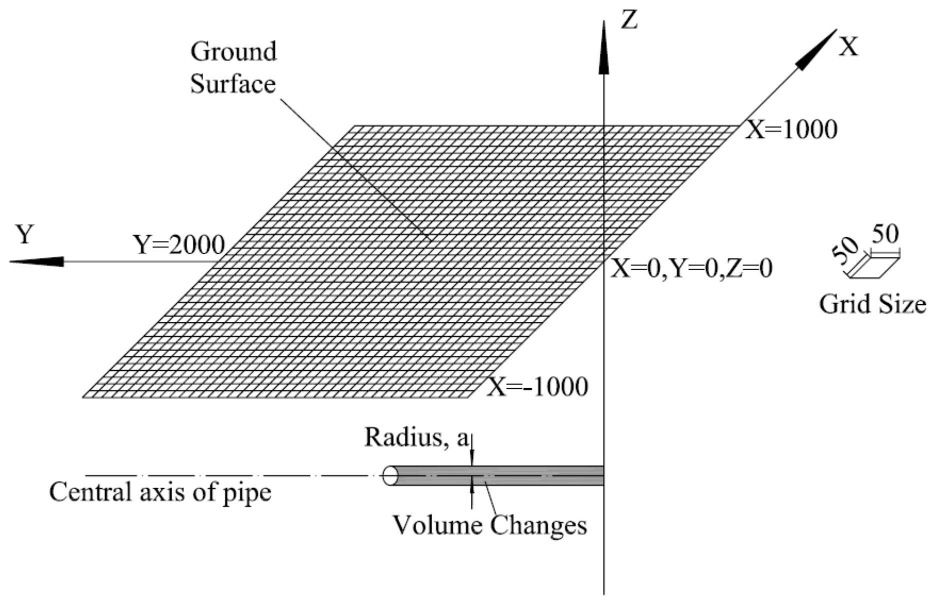

Fig. 4. Sketch of the ground surface grid distribution

A cross-section of predicted ground displacement immediately above the bursting head is shown in Figure 5. A comparison of the predicted and observed surface displacements along the centerline of the host pipe are listed in Table 2. The initial LVDT reading of 24 mm, recorded when the bursting head was 1 m away from the point of measurement, was subtracted from all experimental measurements for proper comparison with the numerical data. It can be seen that predicted values are substantially lower than the experimentally measured values. It can also be seen that the maximum vertical displacement was measured while the bursting head was 250 mm away from the point of measurement, while the model predicts the maximum displacement to occur when the bursting head is immediately beneath the point of measurement.

While there are several possible explanations for the difference between the observed and predicted behaviors (such as ‘block failure’ mechanism during the experimental work), there is little doubt that the numerical model in its current form under estimate the vertical displacement of the soil media for shallow pipe bursting applications. A modification to the current algorithm that better represents the displacement of the in-situ soil during a pipe bursting operation is described in the following sections.

Table 2. Comparison of Experimental and Numerical Values

Distance to burst head, mm	**-1000**	**-750**	**-500**	**-250**	**0**
Predicted vertical displacement, mm	0	1.4	2.07	4.08	4.75
Measured vertical displacement, mm	24	28	38	41	30
Measured vertical displacement (after subtracting initial value)	0	4	14	17	6

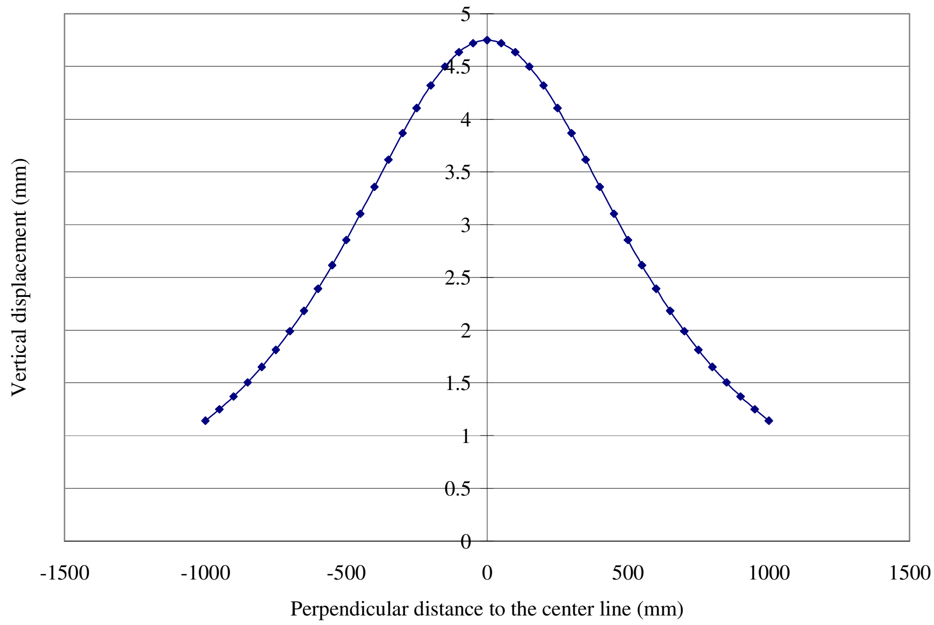

Fig. 5. Vertical displacement of surface immediately above the bursting head

Results and Discussion

The incorporation of an analytical geotechnical ground movement algorithm within ArcView provides a powerful analysis tool for utility and municipal engineers. The designer can new export a shape file that contains pertinent information regarding existing utilities and surface improvements into the analysis module, run the analysis and obtain the predicted soil movement at various locations within the proposed alignment. Using ArcView's spatial analysis capabilities, potential problem areas can be identified with ease.

While relatively simple, in its current form Sagaseta's model cannot accurately capture ground surface movement for shallow pipe bursting installations. The next section presents proposed future work aimed at improving the accuracy of the model's predictions.

Future Work

In the above simulation, volume changes are assumed to occur along the central line of the pipe, which is an over-idealization of the physical process modeled. In reality, the volume changes occur around the host pipe in-situ. Figure 6 shows a sketch of the distribution of these volume changes. Future work will better account for these volume changes by utilizing a series of line sources around the perimeter of the host pipe (rather than a single source along the center line of the host pipe as it was done in the current development).

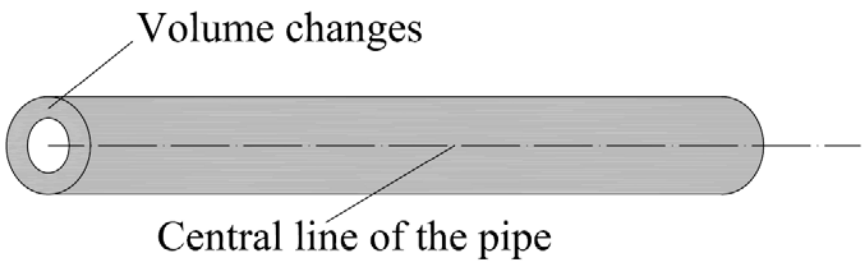

Fig. 6. A schematic of the distribution of the volume changes

Laboratory tests will be conducted at Trenchless technology Center soil-structure interaction chamber to accurately measure and map ground movement during a pipe bursting operations at different levels of in-situ stresses to further validate and improve the proposed simulation model. This testing facility consisted of a 1.2 m high by 4 m long by 2 m wide steel tank and a 150,000 lb servo-controlled actuator which generates the needed static axial force. An air-bladder capable of producing in-situ pressure equivalent of up to 15 ft of soil overburden will be utilized for simulating various buried depths. Using a grid of 20 LVDTs a 3-D map of the ground surface with respect to the position of the bursting head will be constructed to validate the ArcView model. A snapshot of the facility is given in Figure 7.

Fig. 7. TTC Soil Structure interaction testing facility; a) View from the rear including control station; b) Front view including servo-control actuator; c) Overhead crane, top beams and tie-down rods

References

Atalah, A., Sterling, R., Hadala. P., Akl, F., Lei, Y., and Du, X. (1998). The effect of pipe bursting on nearby utilities, pavement, and structures. *TTC Technical Report, No.TTC-98-01*, Trenchless Technology Center, Louisiana Tech Univ., Ruston, LA.

Baumert M., Allouche, E.N., Liu H., Hill D. and St. Aubin R. (2006). Experimental Examination of Belled PVC Pipe Installed Using Pipe Bursting. No-Dig conference, Nashville, TN, March 8 p. (on conference CD ROM).

Chapman, D.N., Falk, C., Rogers, C.D.F., Stein, D. (1996). Experimental and analytical modeling of pipe bursting ground displacement. Tunneling Underground Space Technology, Vol. 11, Suppl.1, pp. 53-68

Duyvestyn, G. (2006). Fairwood interceptor pipe bursting heaves: predictions versus actual. Proceedings of No-Dig 2006, March 26th to 28th, NASTT, Arlington, VA, pp. E-2-04-1—E-2-04-9

Lapos, B.M. (2004). Laboratory study of static pipe bursting, three-dimensional ground movements and pull force during installation, and subsequent response of HDPE replacement pipes under surcharge loading. Thesis for degree of Master of Science in Civil Engineering, Queen's University, Kingston, Ontario, Canada

Saber, A., Sterling, R., and Nakhawa S.A. (2003). Simulation of ground movements due to pipe bursting. Journal of Infrastructure Systems, Vol. 9(4), pp. 140-144

Sagaseta, C. (1987). Analysis of undrained soil deformation due to ground loss. Geotechnique, Vol. 37(3), 301-320

Innovations In Watermain Renewal

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Abstract

Recent advances in minimal-disruption rehabilitation for water systems are increasing the willingness of utility owners and operators to consider these technologies. This trend follows the lead of trenchless sewer rehabilitation methods that have been established as viable options to mitigate the impacts of replacing aging infrastructure. Water system renewal is, however, a more complicated issue with unique requirements. Products must withstand the internal pressures inherent in distribution systems, while being suitable for potable water.

The trenchless technology industry is dynamic, with new options frequently coming to the forefront. In this paper some of the major issues and concerns for consideration when selecting a water system renewal technology are presented and discussed.

This paper focuses on recent advances for lining potable watermains with polyethylene and polyester-reinforced polyethylene (PRP) products in North America, the UK and Europe. It also discusses the implementation of a revolutionary method for internal reinstatement of service connections. Further, the paper reviews the future of water system renewal, discussing outstanding hurdles as well as the next logical steps in industry evolution.

Introduction

Trenchless (or minimal disruption) watermain rehabilitation has been utilized globally for many years. It has been embraced as a feasible option in Quebec, Canada and is now being considered across North America. The potential to renew an aging watermain in a manner that reduces inconvenience, construction time and cost, coupled with identified shortfalls in committed funding for watermain rehabilitation, makes these methods very attractive to utility owners.

Many watermain rehabilitation technologies are derived from existing methods that were developed as solutions for other infrastructure, such as sewers and gas lines. However, watermains have a wider range of requirements than other systems. They must be capable of handling internal operating pressures and be approved for potable water systems.

Selecting the appropriate rehabilitation technology requires knowledge of available techniques, their capabilities and perhaps most importantly, the problem to be solved along with the physical and operating characteristics of the existing pipe.

Liner Classification

In Europe, Committee for Standardization (CEN) and International Standards Organization (ISO) provide convenient categorizations for trenchless and minimal disruption construction methods (Heavens and Gumbel, 1998):

Rehabilitation:	All measures for restoring or upgrading the performance of an existing pipeline system.
Renovation:	Work incorporating all or part of the original fabric of the pipeline by means of which its current performance is improved.
Repair:	Rectification of local damage.
Replacement:	Rehabilitation of an existing pipeline system by installation of a new pipeline system, without incorporating the original fabric.

Separate liner classification systems have been developed in Europe (CEN) and North America (American Water Works Association (AWWA)). Liners that can stand alone to resist the long-term internal stresses are termed "independent" under the CEN system, and "fully structural" under the AWWA system. Liners relying on host pipes to provide some measure of redial support are termed "interactive or semi-structural" under the CEN and AWWA classifications, respectively.

Figure 1, (Gumbel et al, 2004), illustrates independent and interactive liner principles

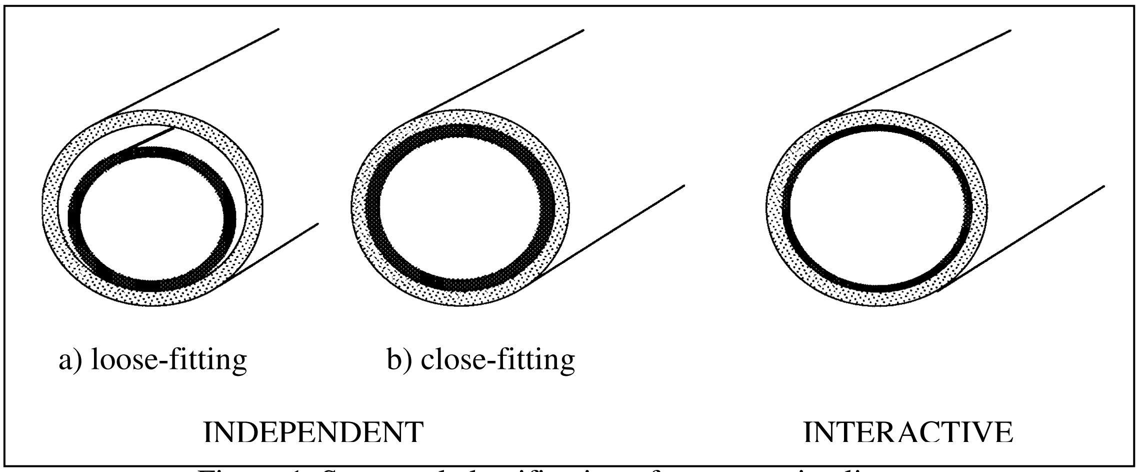


Figure 1. Structural classification of pressure pipe liners
(EN 13689:2002)

The AWWA Manual of Water Supply Practice M28, Second Edition (2001) – Rehabilitation of Water Mains, provides a more discrete definition of four classes of liners:

Class I - primarily protect inner surface of the pipe from corrosion, with minimal ability to bridge discontinuities such as holes or gaps

Class II - long term internal burst strength (tested independently of host pipe) is less than maximum allowable operating pressure (MAOP) of the host pipe; designed to bridge specified size holes and gaps; depend entirely on adhesion to the host pipe wall to prevent collapse if the pipe is depressurized

Class III - long term internal burst strength (tested independently of host pipe) is less than the MAOP of the host pipe; designed to bridge specified size holes and gaps; sufficient inherent ring stiffness to be self-supporting in event of depressurization of the pipe; may also be designed to withstand specified external hydrostatic or vacuum forces

Class IV - long term internal burst strength (tested independently of host pipe) is greater than the MAOP of the host pipe to be renovated; the liner if close-fitting must also be capable of surviving possible future failure of the host

Figure 2 illustrates a comparison of the two classification systems.

<table>
<tr><td rowspan="2">CEN</td><td colspan="2">INDEPENDENT</td><td colspan="2">INTERACTIVE</td><td>-</td></tr>
<tr><td>Loose-fitting</td><td colspan="3">Close-fitting</td><td>-</td></tr>
<tr><td rowspan="2">AWWA</td><td colspan="2">FULLY STRUCTURAL</td><td colspan="2">SEMI-STRUCTURAL</td><td>NON-STRUCTURAL</td></tr>
<tr><td colspan="2">Class IV</td><td>Class III</td><td>Class II</td><td>Class I</td></tr>
</table>

Figure 2. Relationship of CEN and AWWA Liner Classifications

Presently, these classification systems do not consider the implications for possible external failure modes of the host pipe, i.e. longitudinal bending and shear failure associated with ground movements and/or traffic loading. While the existing classification systems address radial stresses, design of pressure liners must also consider the longitudinal forces resulting from pressurization or installation activities.

Design Considerations

Several factors must be considered before selecting a trenchless or minimum-disruption lining method. The impacts of lining on the capacity capabilities of the main, for example, must be assessed. If the lined main provides the required flow, it is then feasible to progress to the next phase. Often, existing mains have some deposits that reduce flow rates. A lined main will have a smaller internal diameter than the host pipe, however, the flow capacity may actually be greater due to lower friction values. The lined main will have a higher flow capacity than the existing corroded line, but it should be noted that it may not provide capacity equal to a new main. If additional capacity is required, then other trenchless technologies such as pipe bursting, horizontal directional drilling, microtunnelling or even tunneling can be considered.

Trenchless construction methods are typically less expensive than typical open-cut projects, however, it is prudent to assess the costs. Many municipalities and infrastructure owners weigh the socio-economic as well as the direct construction costs. Typically, an emergency repair can cost several times as much as a planned project. Similarly, social costs related to construction can also be significantly more than construction costs. When these factors are compounded, trenchless options can be even more economically attractive.

In cases where lining is determined to be a suitable alternative for watermain renewal, there are a number of considerations to assess before finalizing lining option selection. It is important to fully understand the true problem to be solved. Foster and Bontus, 2006, provide a summary of items to address these design considerations:

- Operating Parameters
  - Internal pressure – What is the operating pressure for the main?
  - External load – What load will the liner experience if the host pipe fails?
  - Impact on flow rates after lining – What flow capacity is required?
- Physical Characteristics
  - Mode of deterioration or failure – Internal or external failure?
  - Host pipe wall thickness – Design and existing wall thickness?
  - Effective size of holes and/or joint gaps.
  - Details of protrusions – Services or patches?
  - Existing service and fitting locations – Number and locations of services and fittings.
  - Internal pipe condition – Corrosion or deterioration?
  - Pipe condition at lining termination – The condition of the host pipe may impact selection of termination fitting.
- Installation
  - Cleaning and inspection – The type and amount of build up in the line may impact cleaning methods.
  - Excavation requirements – Determined by the existing fittings and the length of main a lining technique can renew.
  - Temporary water supply – What level of temporary service is required; residential service only or fire protection as well?
  - Reinstatement of services – Number and size of services?
  - Valve and fitting replacement – Renovated watermains typically have a design life of 50 years plus; existing fittings and valves will likely require replacement to meet these design conditions.
  - Site footprint – Various renewal methods have different site footprint requirements, and not all may be suitable for all applications.

Pipe Renovation Solutions

Liners can be categorized into four families. Table 1 presents a summary of their characteristics and some typical products. Subsequent sections will expand on several of these options.

Table 1 – Liner Families

Liner Type	Characteristics/Parameters	Typical Products
Coatings	• Lining is typically carried out with spray-on protective coatings applied inside existing pipes. • Water quality problems are the primary issue, and the host pipe has structural integrity. • Water chemistry and durability requirements drive design. • Typically designed as Class I liner, although it may be possible to achieve Class II in high build-up applications.	o Cement mortar o Epoxy o Polyurethane
Continuous Pipe Liners	• Lining is carried out with a continuous pipe the length of the section to be renovated. • Liner pipe is fused or joined prior to lining. • Cross section is not modified. • Does not rely on host pipe, therefore designed as Class IV	o PE o PVC
Close Fit Liners	• Lining is carried out with a continuous plastic pipe of the length of the section to be renovated; the entire length of liner may be fused prior to installation, or it may be fused as sections are inserted. • Liner pipe has a reducing cross-section, either radially or by folding. The liner may be: o Site reduced o Factory reduced • Designed as Class IV or Class III, depending on material selected and project requirements.	• Typical pipe types: o PE o PVC • Example site reduced products: o PolyFold™ o PolyFlex™ o Swagelining • Factory reduced products: o Thermopipe™ o U-liner™ / Compact pipe o Duraliner™

Cured-In Place Pipe (CIPP) Liners	• Lining is carried out with a continuous, resin-impregnated flexible tube of the length of the section to be renovated. • Types of CIPP product: o Resin-fiber composite pipes o Adhesive-backed hoseliners • Depending on the application, composites may be designed as Class IV or III; hoseliners generally as Class II.	o Insituform PPL® o Nordipipe o Aqua-Pipe™ o Tubetex

Site-Modified Polyethylene Renovation for Municipal Systems

The practice of lining existing pipelines with site-modified PE has been used for many years. Two primary methods for modifying the cross-section of the liner are radial reduction, and some form of folding the liner.

Radial Reduction Process

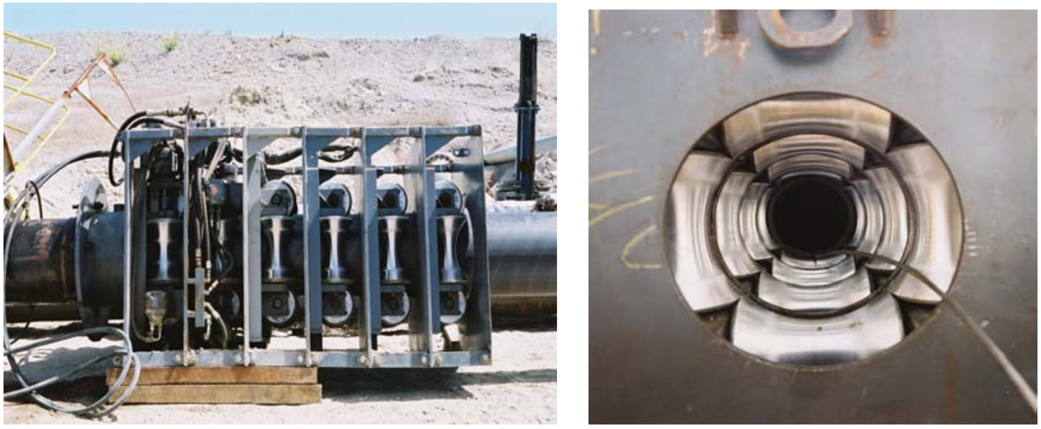
In the past, companies have used radial reduction to install tight-fit liners in pressure pipelines from two to 54 inches in diameter. For example, over 11,000 km of United Pipeline Systems' Tite Liner® has been installed in pressure pipes throughout the world. It is typically designed as an interactive liner, and is therefore often in the DR 32 to DR 40 range.

The interactive radial reduction liner system uses a radial reduction machine to reduce the pipe diameter by approximately 10 percent, while pulling the liner into the host pipe using a winch. Once the segment is lined, the liner is allowed to relax, reverting to its original diameter, or to a tight fit against the host pipe.

For municipal water systems, a new installation technology has emerged. This radial reduction process (RRP) uses similar radial reduction machines, but may reduce the pipe diameter by as much as 20 percent. A greater reduction in liner diameter reduces the amount of tension required to insert the liner, and consequently, the size of winch. This also means that the liner is stretched less, and the amount of "rebound" stress in the pipe is reduced. RRP results in minimal pull on the pipe, relying largely on the driven rollers in the radial reduction machine to push the pipe into place.

Once the liner is in position, it can either be allowed to re-expand on its own, or through pressurization with water. The liner is then terminated at each end with a coupling suitable for connection to the existing system.

RRP provides either an independent or an interactive liner, depending on the project requirements. In municipal water systems, it is applicable in runs of 1000 feet, for liners ranging from DR 40 to DR 21, and sizes up to 24 inch.



Radial Reduction Machine

Folded Pipe Process

The same company that developed RRP has also developed a system that folds PE pipe into a "celery" shape for insertion into an existing pipe, resulting in a close fit liner. This folded process is applicable for watermains from 12" to 48" and greater and liner wall thickness ranges up to DR 21. In this process, a circular PE pipe is inserted into the folding machine, where the cross-section is modified (with the cross section reduced by up to 40%) using a series of rollers to guide the pipe. The machine consists of a hydraulic pushing component, as well as the roller section. Once the pipe exists the machine, it is banded to retain its modified shape.

This folding process offers options for installing the modified pipe. Either the entire length of pipe can be fused, modified and then inserted into the host pipe, or the machine can be set up at the insertion point, and the pipe can be fused and fed into the machine on a piece by piece basis. This second option results in a significantly smaller site footprint.

The liner is pulled into the host pipe via a winch. When the liner is in position and cut to length, the bands on the exposed ends are cut, and the pipe is re-rounded at these locations with Kevlar reinforced airbags. The selected end couplings, with the same options as for the radial reduction process, are attached along with blind flanges.

The liner is then pressurized using a two stage water process. First, restraining bands snap at 10 – 15 psi, then the pipe is pressurized further to re-round the full length.

The folded process can provide either independent or interactive liners, depending on the project requirements.

Photos: Process close-up, folding machine and banding, and installation

PE Liner Capabilities

New PE resins and processes are improving liner performance. PE 100, an ISO designation, and ASTM 4710 pipe offer higher pressure capacities than traditional PE 80 or 3408 pipes. They also offer greater resistance to rapid crack propagation.

If a host pipe has sufficient structural integrity, an interactive liner can provide a long-term solution without significantly impacting the flow capacity of the line. The properties of PE allow interactive liners to span significant holes in the host pipe. Depending on the maximum system operating pressure and liner pressure capabilities, an interactive liner can often span significant holes in the host pipe without failing.

Polyester-reinforced Polyethylene (PRPE) Liners

PRPE is a factory-folded liner that can independently accommodate pressures of 170 to 230 psi. Woven polyester fabric is encapsulated within a polyethylene coating, meeting ANSI/NSF 61 requirements. The liner is folded into a tight, flat "C" shape, and wound on reels for transport. PRPE is available in sizes from 2 ½ inches to 12". Over 500,000 feet of Thermopipe™ have been installed in the UK and Europe.

PRPE can be installed rapidly, through relatively small access pits. Installation of over 1000 feet of liner in one day is possible. The liner is pulled through the host pipe using a small winch and inflated using air pressure. While still under pressure, steam is introduced into the liner for approximately 25 minutes after which the liner is cooled (still under pressure using air). The entire process requires only 15 psi, and does not involve any chemical curing.

Flanged Redman fittings (compression-type fittings, installed using a hydraulic hand pump producing 3500 psi) are typically used to terminate all lined segments.

Inserting Folded Liner and Expanding Liner

Internal Service Reinstatement

Perhaps the most significant advance in watermain renewal is the ability to reinstate service connections from within the lined pipe, thereby reducing the impact on residents and traffic. For several years cured-in-place pipe (CIPP) liner contractors (primarily in Quebec, Canada) have reinstated services by drilling out resin that entered existing main stops during the lining process. Knight and Sarrami (2006), carried out independent third party testing on Sanexen's Aquapipe, a CIPP lining system, and reported, among other findings, that a suitably saturated liner would bond to the host pipe, with small voids possible around the service connections.

The mechanical internal service reinstatement technology was introduced in 2006. It uses self-tapping brass fittings with rubber gaskets to form the seal between the existing main stop and the liner. The system is designed for use with both PE pipe applications and polyester-reinforced polyethylene (PRPE) liners.

The mechanical service reinstatement process is simple and straightforward. Prior to lining, all services are milled flush with the host pipe using a remote controlled robot. After the pipe is lined, a remote eddy current sensor is used to locate the main stop. Another set of robots remove segments of the liner at the service location, and then inserts the left-hand threaded, self-tapping fitting.

Fittings for mechanical service reinstatements are available for a range of services sizes, and expansion of the range is underway. Currently, the system can be used in lined pipe ranging from 6" through 12", and equipment for other sizes is being developed.

Completed reinstatement

Future Trends

Trenchless watermain renewal offers many benefits over other construction methods, in appropriate applications. In such a dynamic industry, options and capabilities continually evolve.

Currently in the UK, renovated water systems are reinstated in the same day. This is possible in some areas due to the acceptance of the steam used in the PRPE lining process, which provides disinfection. In other locations, a temporary boil-water order is issued until test results are received. In North America this is rarely possible, however, efforts are underway to address reducing the time that systems are out of service.

The present envelope for lining technologies is continually being expanded. Numerous feasible options to renovate standard sized distribution systems are available, i.e., 6” through 12”. Options for transmission systems are, however, limited due to the pressure requirements and structural issues. Ongoing research and development is focused on providing large diameter, high-pressure liners.

At the other end of the spectrum, there is a huge interest lining smaller water pipes. In parts of the UK and Europe, domestic and fire water systems are separate. Reinstating services in smaller diameter water mains would make trenchless renewal even more viable. Stretching the envelope farther, the means to line actual service lines without whole scale digging would allow full system rehabilitation with minimal disruption. As these issues are addressed, opportunities for trenchless watermain renewal will continue to grow.

Bibliography

AWWA Manual M28Rehabilitation of Water Mains, 2nd Edition. American Water Works Association, Denver, CO.

Brand, M. 1999. Relining of Pressure Pipes, Expanding the Envelope. Proceedings of the Joint Symposium of the GSE and the Northwest Chapter of the NASTT, Edmonton, Alberta.

Foster, K and Bontus, G 2006. Emerging Technologies in Watermain Renewal; Proceedings of INFRA 06, Quebec City, Quebec.

Gumbel, J. E. 1998. Structural Design of Pipe Linings 1998 – Review of Principles, Practice and Current Developments Worldwide. Proceedings of the 4th ASTT Conference, Brisbane, Australia.

Gumbel, J.E., Elzink, W. and Heavens, J. 2004. The Rehabilitation of Pressure Pipelines: Key Issues in the Design and Selection of Renovation Technologies. ISTT Workshop, International NO-DIG, Hamburg, Germany.

EN 13689:2002 “Guidance on classification and design of plastics piping systems used for renovation”, CEN (European Committee for Standardization), Brussels, October.

Knight, Mark and Sarrami, Kamran (2006) “Third Party Evaluation of the Aqua-pipe Watermain Rehabilitation Product”, Proceedings of NODIG 06, Nashville, TN

Pneumatic Pipe Ramming Solves Emergency Situation for Rail Corridor

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Background

Pipe ramming continues to be one of the most versatile and capable trenchless pipe installation methods. The method is proven effective for horizontal, vertical, and angled applications. In recent years, the method has set new records in large diameter casing installations and is becoming more and more common.

Pipe ramming is a favorite installation method for contractors installing casings under roads and rail lines because the method provides accurate installation in a wide range of soils without surface slump.

Ramming Basics & Benefits

Trenchless pipe installation through ramming is a basic process that can have amazing results. A pneumatic hammer is attached to the rear of the casing or pipe. The ramming tool, which is basically an encased piston, drives the pipe through the ground with repeated percussive blows.

A cutting shoe is often welded to the front of the lead casing to help reduce friction and cut through the soil. Bentonite or polymer lubrication can also be used to help reduce friction during ramming operations.

Several options are available for ramming various lengths of pipe. An entire length of pipe can be installed at once or, for longer runs, one section at a time can be installed. In that case the ramming tool is removed after each section is in place and a new section is welded onto the end of the newly installed section. The ramming tool is connected to the new section and ramming continues. Depending on the size of the installation, spoil from inside the casing can be removed with compressed air, water, an auguring system or other types of earthmoving equipment.

Some casing installation methods are impaired or even rendered inoperable by rock or boulder filled soils. Pipe ramming is different. During pipe ramming, boulders and rocks as large as the casing itself can be "swallowed up" as the casing moves through the soil and can be removed after the installation is complete.

Ramming tools, in general, are capable of installing 4- through 122-inch diameter pipe and steel casings. At 24 inches in diameter, the Grundoram Taurus is the world's second largest pipe rammer. The Grundoram Apollo at 32 inches in diameter is the world's largest ramming tool.

On the Job

Recently, the Union Pacific Railroad discovered significant drainage problems after heavy rain and snow melt at its main east-west rail corridor through northern Nevada and northern California along the Feather River Canyon. Culverts that should have been able to allow the runoff water to pass safely under the deep cuts had failed, collapsed etc. Emergency crews worked around the clock pumping water up and over the steep embankments to prevent rail line washout.

Rail traffic was slowed in order to prevent additional damage to the saturated soils of the embankments or rail failure. The Union Pacific enacted emergency plans and hired a contractor to install new culverts.

Pipe ramming was chosen to install the new 48- and 60-inch culverts to relieve the water pressure. Ramming was ideal because the canyons were full of rocks and boulders. A 24-inch diameter pneumatic pipe ramming tool was used. Access was difficult. Materials were transported by railcars and then lowered 65 feet by crane.

The Union Pacific and other railroads have many old culverts that are in this condition or worse. A case can be made that the pipes supporting rail traffic are in need of rehab and replacement as badly as other decaying infrastructures like old gas mains, sewers and water mains.

Case Studies for a Free-Swimming Acoustic Leak Detection System used in Large Diameter Transmission Pipelines

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Introduction

Acoustic leak detection systems have proven to be an effective means for identifying leaks in pipelines. Recent technological advances have resulted in the development of a free-swimming acoustic leak detection device capable of surveying many miles of pipeline with a single deployment into a fully operational pipeline. The system can identify and locate small leaks in water pipelines larger than 10" diameter constructed of any pipe material. The device can also pass through in-line valves (including butterfly valves) and negotiate unlimited bends in the pipeline. These benefits provide a cost effective and timely approach to identifying and locating leaks in large diameter transmission pipelines.

Background

Acoustic leak detection equipment identifies the sound or vibration induced by product (i.e., water, oil, gas, hydrocarbon product, etc.) escaping from pipes under pressure. When pressurized product leaks from a pipe, it creates a distinctive acoustic signal that travels through the product flowing in the pipeline.

Recognizing the value offered by acoustic leak detection technology, yet realizing some of the limitations associated with current leak detection technologies applicable to large diameter water transmission mains, a research and development program was undertaken by Pure Technologies in 2004 to develop a free-swimming (non-tethered) acoustic leak detection device. The goal of the program was to develop a technology that would provide operators and engineers with the ability to survey pipelines for which inspection was previously was either logistically challenging or not possible. The free-swimming leak detection device was designed not to replace existing leak detection technologies, but instead to compliment them.. This allows an operator the opportunity to select from several different technologies to best address their needs.

As stated, the research program focused on leak detection in larger diameter water transmission mains. Typically, such water mains run for long distances and do not offer much in the way of intermediate access points. Hence, a free-swimming leak-detection device was developed that could be propelled with the water flow over long distances while recording acoustic signals generated by leaks.

In developing this device it was recognized that the advantage of having the sensor pass very near the leak (no further than a pipe diameter), provided a highly sensitive leak detection method. One of the major challenges in designing the device was to provide for the sensitive detection of the acoustic signal generated by a leak, with minimal interference from noise generated by the movement of the device as it traverses the pipeline.

Water Pipelines vs. Oil Product Pipelines

For water pipeline applications, the final design of the free-swimming device consists of a foam ball that envelops a water-tight, aluminum sphere (approximately 2-1/2" in diameter) containing the sensitive acoustic instrumentation. The device is inserted into the pipeline and released to allow the flow to carry it downstream. While the ball is traversing the pipeline, it continuously records all acoustic activity in the pipeline.

Timer boxes are placed periodically along the pipeline at convenient surface access locations, whereby the movement of the device can be tracked. Once the ball has traversed the desired pipeline length, it is retrieved from the pipeline. The acoustic data can then be evaluated on site to determine the presence and location of any leaks in the pipeline.

For water pipelines, the technology requires two flange openings at least four inches diameter with a full port valves (one each for insertion and extraction of the device). Once deployed, it can move through in-line valves (including butterfly valves), through reducers and other fittings, as well as navigate unlimited bends and profile changes. The ball is propelled by the hydraulic flow of the water and will roll along the bottom of the pipeline. As long as the flow rate is below 3-1/2 fps, the ball will roll, and the accelerometers in the instrumentation package will be able to bench mark the distance the device has traveled at any given time.

The current configuration of the device provides for thirteen hours of acoustic monitoring and recording. Based on a flow rate of two feet per second, the free-swimming leak detection device would be able to survey approximately fifteen miles of water pipeline with a single deployment.

Even if a water pipeline were to extend beyond 15 miles, there is no need to install additional 4" outlets at 15 mile intervals to accommodate insertion and retrieval of the smaller device along the pipeline route. The microprocessor in the instrument core can be programmed with a delay so it only begins recording after a certain time interval, allowing the user to inspect the entire line by means of multiple runs. The data from the runs would be set to overlap to ensure coverage of the entire pipeline.

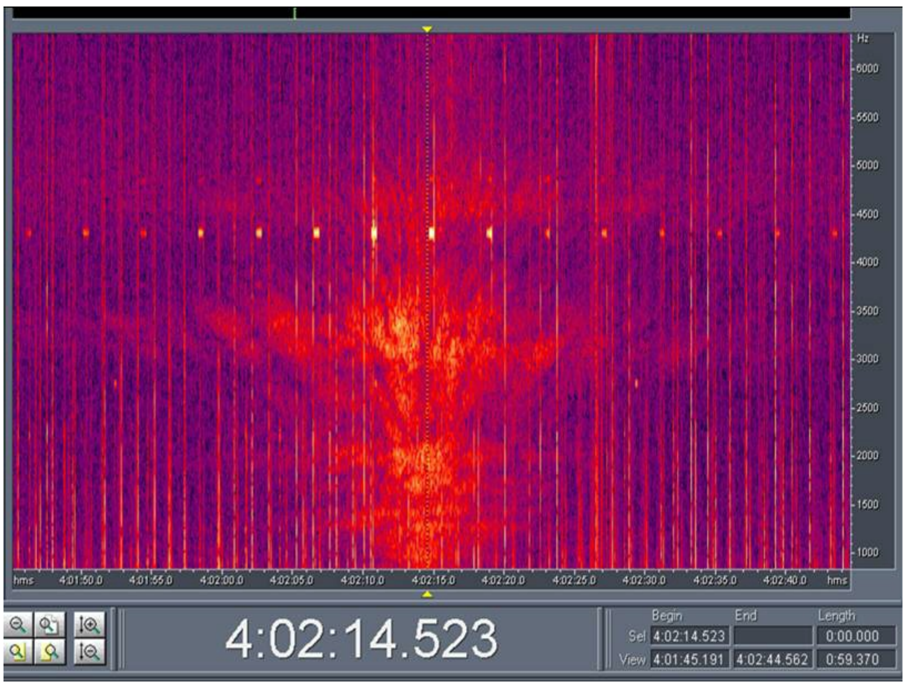
Oil product pipelines typically operate at much higher pressures and greater flow rates than water transmission lines, and are typically much longer as well. The electronics canister is thus designed to withstand up to 3000 psi, and the protective outer ball is made of a more rugged polyurethane material.

Because oil and gas product pipelines are typically equipped with full diameter pig launching and receiving facilities at each end of the pipeline, the size of the device was increased to an overall 9” outer diameter. This allows for a larger aluminum core to hold the acoustic instrumentation and accommodate additional battery capacity. In the current configuration, the core is a 5-1/2 cylinder that is 5-1/2” long and provides a 48 hour battery life. A longer life version is being designed which would have over a week of battery life. At a flow rate of 5 ft/sec, the device would be able to traverse over 600 miles in a single deployment.

The larger device could also be utilized in water pipelines, however, the device would require a 10” flange opening and full port valve to accommodate insertion and retrieval of the device.

Identifying and Locating a Leak

Initial testing of the device in 2006 confirmed its ability to detect leaks as small as one-tenth of a gallon per minute. A print out of an audio frequency representation for a one (1) gallon per minute leak in a water pipeline is presented below.



The technology utilized in the free-swimming leak detection device uses several features in combination to accurately locate a leak in a pipeline. These include:

- A miniature transponder placed inside the sphere emits a coded ping that allows a GPS based logger or timer box on the surface to constantly monitor the position of the device.

- Detecting pulses from positioning transponders located at known locations on the surface. Transponders can be periodically placed along the pipeline that time-stamp the ball as it is traveling.

- Counting revolutions. It has been observed that in a flow less than 3.5 ft/sec the ball rolls along the bottom of the pipe without skidding or any extra revolutions. Using the accelerometer data to monitor the rolling provides a general indication of where a leak is or to confirm other locating algorithms.

- Counting metallic joints. In certain types of pipe like PCCP a magnetometer is used to "count" pipe sections by detecting steel joint such as metallic joint rings and other steel appurtenances such as valves, side outs and steel fittings. In a steel pipeline, the location of welds can be detected by means of the accelerometer data.

- Internal monitoring and recording of temperature and temperature changes can denote inlet/outlets along the pipeline.

Case Studies

Note: Pure Technologies is currently in the process of deploying the non-0tethered leak detection device in several pipelines for major water agencies and oil product pipeline companies. Results of the leak detection results should be available before the time the final paper is due.

Lessons from the Investigation of Problems in Airport Way Sanitary Sewer in Portland, Oregon

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Abstract

City of Portland-Bureau of Environmental Services (BES) contracted with BC Construction Company (BC) to construct the NE Airport Way II-NE 138th Avenue to NE 181st Avenue Water and Sanitary System. The contractor did not complete works to the satisfaction of the city. The city was subsequently forced to incur additional expenses to bring the line to acceptable condition before putting into service. The city wanted to recover the added cost from the Contractor, and engaged the author as an expert to review the contract documents, reports and records to find out whether it had the foundations and technical basis for its case and legal action against the contractor. The author completed the investigation and this paper presents a summary of the lessons learned from this investigation.

Background

NE Airport Way Phase II NE 138th Ave to NE 181st Ave Water and Sanitary System is part of City of Portland Local Improvement District Projects. The city had done extensive geotechnical investigations for the project to identify construction related problems such as dewatering, shoring systems, and for the selection of suitable bedding and backfill. Geotechnical Investigation Reports were included in the bidding documents and in turn in the Contract documents by the city, calling attention to on-site conditions, in particular about the presence of boulders and excessive groundwater.

Scope of Work

The scope completed by the author for the City's Attorney included the following tasks:

a) Review the conditions of the line prior to its recent replacement;

b) Review for appropriateness of the City's design for the pipeline in terms of the choice of pipeline materials and the existing soils conditions;

c) Review the work performed by the road contractor and determine whether it could be the cause of the pipeline problems;

d) Determine the exact date and magnitude of the most significant earthquake in the area and determine whether it could have been the cause of the problems;

e) Work with City staff and review City materials as necessary for the work;

f) Provide City staff with information regarding how to review and evaluate a claim of this nature;

g) Collect data on the performance of similar jobs "in the area" by other contractors.

h) Evaluate the contract specifications, dailies, the design documents, change orders, correspondence, videos, and photographs;

i) Examine any other issues which are relevant to the case.

The author completed the above scope and helped the city reach a settlement with the contractor rather quickly.

Geotechnical Reports

The site was investigated by several geotechnical consultants engaged by BES, in order to characterize the site and to evaluate possible construction issues relating to the construction of the Airport Way Phase II Sanitary Sewer Line. Several studies and reports that were reviewed for the evaluation and were available to BC at the time of bidding are as follows:

- Geotechnical Investigation, Proposed NE Airport Way Interceptor and Pump Station, Dames and Moore, August 1985
- Geotechnical Report for NE Airport Way; I-205 to I-84, Geotechnical Resources, Inc., July 1986
- Supplemental Geotechnical Studies, NE Airport Way Interceptor Sewer, Geotechnical Resources, Inc., October 1986
- Additional Geotechnical Studies, NE Airport Way Interceptor Sewer, Geotechnical Resources, Inc., October 1986
- Consultation for NE Airport Way Water Quality Protection, Geotechnical Resources, Inc., June 1989
- Groundwater Supply Main Geotechnical Exploration, CH2M-Hill, Jan 1990
- Geotechnical Exploration, Airport Way Slough Crossings, R. Henhouse - Zeman & Associates, Inc., April 1990
- Geotechnical Engineering Services NE Airportway Sanitary Sewer Phase II and III, Dames and Moore, October 1990

These reports provide information on site investigations and site evaluation. Prior to any work by BC, a roadway embankment had been built to surcharge the project site and to induce primary consolidation of the alluvial soils. This is normal practice among experienced geotechnical engineers to make sure that the construction-induced settlements afterwards are minimal. Subsurface conditions at the site consist of alluvial very soft to medium stiff sandy and clayey silt to a depth of 24 feet. The sandy soils were characterized by relatively low strength and moderate to high compressibility. The deeper gravelly soils were characterized by relatively high strength and low compressibility. Very high groundwater elevations were also reported. In order to counter the possibility of foundation instability due to bottom heave, piping due to high ground water elevations, and hydraulic gradients, the City engineers specified foundation stabilization and special pipe bedding procedures. The City engineers expected that proper rock stabilization and bedding procedures would be followed by the contractor. Suitable trenching and lateral support methods were also recommended in the reports and were identified in the contract documents.

BES incorporated the Geotechnical Engineering Consultants recommendations in the design of the sewer lines and in the construction specifications, clearly indicating the requirement for a proper dewatering system to control ground water during construction in poor ground conditions. After being advised by the City of the concerns regarding the groundwater table, BC brought in their expert. BC's list of references of past projects indicated several of similar nature. However, there was no indication whatsoever, about BC's expert's involvement in any of these past projects completed by BC. BES was concerned about the dewatering efforts by BC for details of their proposed methods of handling groundwater and the notes from all of the pre-construction meetings have recorded these really well. It is also interesting to note that no reference is made to either any approvals of a plan by the City engineers or whether an approved plan was followed through in the field. In the author's opinion, the dewatering problems continued due to improper dewatering means and methods used by the contractor. Adequate pipe bedding conditions were not achieved by the contractor due to poor dewatering practices. And the above construction problems can be inferred from the cracks observed in the pipes and joint separations.

Review of Contract Drawings & Specifications

Review of contract drawings, specifications, and other documentation clearly indicate that the city provided adequate information to the contractor regarding the ground conditions, pipe design, and installation requirements.

Review of Project Correspondence & Construction Reports

Review of BES correspondence and construction inspection records were carried out to check contractor compliance with contract documents. Failure of Barnard's dewatering system, damage and service interruption of utilities during construction was also documented. Construction reports also indicate soil profiles from BES inspectors' visual observations at various stations along the sewer. The inspectors denote the soils as "clays" which are reported as clayey silts or clayey sands in the geotechnical reports. When clayey silts or clayey sands are wet, the cohesive behavior is exhibited, and this may have led to the inspectors recording these soils as clays. Soils in this condition are less permeable and very difficult to compact, particularly if flooding was used as a method of compaction as in this case. Compaction test results indicate lower compaction with depth leading to more problems with pipes installed deeper. Construction reports also indicate, poor pipes delivered by the pipe manufacturer, pipe breakage during installation, replacement and repair of pipes, joints failing, and excessive leakage all during construction. Furthermore, results of compaction tests indicate densities varied with depth of backfill settled by water. Higher densities were observed at shallow depths and lower densities at greater depths indicating poor compaction effort in the pipe zone. Repairs done during construction, as indicated in the construction reports, included concrete collars poured directly on the pipe barrel without additional foundation to support this extra weight contributed to cracking of the pipe.

Review of Similar Construction Contracts

Several other contracts in the vicinity of this project were reviewed carefully for design and construction details. It was rather interesting to note that in all other contracts similar RCP were used and due to superior construction procedures, the pipelines did not suffer either excessive cracks or leaks.

City of Portland's Position

A careful review of the nature and the frequency of cracks suggest several factors causing these cracks. These factors are discussed in the remaining sections of this paper.

Improper Handling of Groundwater (Dewatering)

The most significant factor leading to the problems on this line is the lack of adequate methods to collect and dispose groundwater during pipe laying, bedding and backfill placement, and compaction. The contract specifications used by the City for bidding the project provided ample warnings to all prospective bidders about the presence of large amounts of groundwater, mixed soils, and large boulders. BC used means and methods for handling the groundwater which fell far short of what would be effective for a site with so much groundwater. Most of the water was picked up by the simplest, cheapest, and least costly sump pumping method, but unfortunately this method simply is a very poor choice for this site. Even when wells were used for lowering the groundwater, their use was for minimal amounts of time and at very few locations, which in essence amounted to almost no wells at all on the overall effort to lower the groundwater table. A direct correlation exists between the frequency of cracks and the low points along the alignment of the pipeline, where the contractor faced the most quantities of groundwater.

Review of Pipe Design

Reinforced concrete pipes (RCP), and pipe bedding for various applications are designed on the basis of the guidelines provided in the American Society of Testing Materials Standards (ASTM). These standards are periodically reviewed and updated by experts, but primarily controlled by the concrete pipe industry. The bedding used for the project was ASTM C-76 class B. The maximum trench-width allowed was 6.1 m (20 feet). The classes of pipe used in this project were III, IV, and V, where Class III being the lowest D-load crushing strength. The factors of safety calculated for this project for the designs used by the City engineer were all higher than 1.5, which is 50 % more conservative than the industry advocates for reinforced concrete pipe. Thus, the design used by the engineer was more than adequate for the ring loading from Marston's design method. Marstoṇ's design method is applicable to rigid pipes such as Concrete Pipes and has been in use for several decades.

Defective Pipes used

BC had a major problem with the quality of the pipe supplied. This has been extensively documented in their daily reports and in the correspondence between the pipe supplier and BC. There is a very strong possibility that the defective pipes had been installed without being detected prior to installation. In fact, there is a strong correlation between the repaired sections and locations where defective pipe was delivered on the project.

Construction Techniques

It is the author's professional opinion that the contractor fell short in a number of ways and each of these are outlined for us to learn some lessons to remember for the future when we deal with similar pipeline projects.

The City allowed the contractor to use foundation stabilization material below the pipe in poor ground, and agreed to pay the contractor on a basis of unit price times the volume of such foundation stabilization material consumed. The contractor apparently took advantage of this additional open-ended revenue. Rather than provide adequate groundwater collection and disposal methods to prevent soggy bottoms in the foundation, the contractor used large quantities of stabilization rock with an additional cost to City while saving money on the fixed price contract on dewatering systems. Even with this additional expenditure, the City did not receive properly prepared trench because the dewatering system used by the contractor was inadequate. During construction the contractor lowered the trench box into place, and after excavating the soil beneath, extended the flexible sheets beyond the rigid toes of the box. Due to excessive groundwater and lateral earth pressures, these extension plates would became bent toward inside of the trench cutting into the amount of bedding and foundation stabilization that was available for the pipe to support the earth loads. Reduction in the width of good quality bedding material surrounding the pipe would cause the load supporting capacity of the pipe to become lower. The imported material was dumped and pushed down by the bucket of the bulldozer. This was all what BC did to prepare the trench and the foundation, lay the pipe, and place the bedding material. And this is below the "standard of care" that should have been afforded by BC on this project, given the difficult site conditions.

Although the pipe is normally designed in the circumferential direction, all pipelines have a tendency to bend in the longitudinal direction and experience some bending stresses. The manner in which this is handled normally is for the contractor to excavate bell holes adequately so that the pipe is supported primarily along the barrel only. The uniform support along the barrel of the pipe is possible only if the water in the foundation is controlled properly and when the imported bedding material or foundation stabilization rock is placed uniformly and compacted properly. Uneven bedding leads to bending in the longitudinal direction and cracking of the pipe in the circumferential direction and this has been one of the factors contributing to the cracks. The simplest manner in which the stresses from uneven bedding could be carried is to provide additional steel reinforcement in the pipe wall in the longitudinal direction. The only requirement for longitudinal reinforcement steel in the concrete pipe standard ASTM C-76 is to hold the circumferential reinforcing cages in place and this is not sufficient in situations where the pipe goes into bending along its length. But the City's specifications met ASTM C-76 and correctly indicated to the contractor that special dewatering methods were required to provide even bedding. It is important to point out that the pipes that were made to meet even the minimum standard on this project had serious problems and were returned back to the pipe supplier by BC during various stages of the project.

The City Inspectors' reports indicate the site contained many large boulders. This conforms to the warnings in the bidding documents provided to the contractor. The contractor may have been negligent by not removing several of these boulders which were close to the bottom of the pipe during excavation of foundation preparation. If the concrete pipe rests on these boulders, which in essence are hard spots, the pipe would experience point loads, which in turn would cause the pipe to crack.

Barnard's Position

The BC's dailies indicate that the City inspectors were responsive to BC's requests regarding the amount of the Foundation Stabilization Rock (FSR) used. Although from the cost perspective City had an interest to limit the amount of foundation stabilization used, there is no evidence that BC disagreed with City inspectors over this issue at the time of construction.

Sewer pipe materials fall broadly into two types namely, rigid and flexible. RCP, vitrified clay, and some thick-walled ductile iron would be called "rigid", while HDPE, PVC, Fiberglass would be called "flexible". Rigid pipe materials rely mostly on the inherent strength of the pipe to support earth and live loads, while flexible pipe materials need good bedding systems and construction techniques to carry the same loads. Given the groundwater conditions at this site and if the contractor had followed sloppy construction methods similar to those used on this project, another pipe material such as HDPE or PVC would have led to buckling failures and collapses. HDPE and PVC are flexible pipe materials and these rely to a greater degree on the bedding system and desirable native soil conditions to develop adequate stiffness to support external soil and groundwater loads. Therefore, the concrete pipe would still be an appropriate pipe material provided the pipe supplied by the local vendor met the requirements of the project. City Inspectors' reports indicate that broken and damaged pipes were delivered to the site, which were either rejected or field repaired, with instructions to the pipe supplier to improve quality control during pipe manufacture. Even when the pipe was repaired in the field, it was done mainly with either a chemical grout or quickset, which might have resulted in pipes weaker than those with no defects delivered by the pipe supplier. There is a very strong possibility some of the pipes used on the project did not meet the requirement of the project. The bedding materials specified for this project are adequate for the site conditions and pipe loadings.

Seismic Considerations and Impact of Earthquakes in the Area

City of Portland is vulnerable to earthquakes. These earthquakes can be one of the following three types:

- Crustal Earthquakes
- Interplate earthquakes
- Great subduction earthquakes

The Scotts Mills Earthquake of March 25, 1993 is a crustal earthquake. Most earthquakes to date recorded in Oregon fall under this type. Maximum magnitude of crustal earthquake expected in the area is estimated at 6.5. Oregon Department of Geology and Mineral Industries (DOGAMI) has reported that no ground rupture, cracking, land sliding, liquefaction or other surface geologic effects could be reliably attributed to the Scotts Mills earthquake. The peak acceleration due to the Scotts

Mills event in Portland is reported as 0.03 g. This would not cause any structural damage to buried pipes with adequate bedding and side support. Review of the videos taken of another project, Airport Way Phase I, show no signs of any cracks in the RCP although this pipe was loaded by the same earthquake which shook the cracked pipes in Airport Way II. In summary, the earthquakes in the area had no appreciable impact on the concrete pipe used in the projects.

Other issues raised by BC

The depth of fill over the sewer pipe ranged from 2.74 to 6.10 m (9 to 20 feet) and the effects of road construction equipment on the concrete pipe were minimal and did not contribute to the excessive cracks in the concrete pipe. Surface loads such as traffic loads decrease in intensity with depth as they affect the pipe stresses, and become less significant with depth, and this aspect is now used in the design of pipes universally. The factors of safety calculated for the pipe in ring bending when the road construction equipment loadings were taken into consideration ranged from 1.5 to 1.9. There may have been some soil migration given the native soil types and the bedding materials placed by the contractor. However, this could not have caused the circumferential cracks in the pipes. When soil migration takes place, the pipe has a tendency to settle evenly along its length leading to low bending stresses in the axial direction.

Lessons Learned

There are many lessons learned from this dispute:

1. Quality is defined as meeting the project requirements. The QA/QC program begins with the conceptual stage and continues till project commissioning. All work shall be reviewed to accomplish the goals of the project.
2. Engineering designs performed within the scope of any design contract are based on reasonable assumptions. The designers attempt to select reasonable design parameters based on good engineering judgment keeping the safety of the public and the economics of the subject project in mind. The success of the project will rely rather heavily on the abilities of the pipe manufacturer to provide good quality pipe, the contractor to install the pipe properly, and the inspectors to provide reliable QA/QC measures in the shop and in the field.
3. In summary, the designs will work well provided that the design assumptions are met in the shop and in the field. And, unless work is checked for compliance with the engineering specifications for the pipeline project, owner and their representative will never know whether what they are getting is indeed what was specified in the contract documents. If the owner does not plan to allocate sufficient resources to check for the compliance, then there is no point in even spending the resource to write good engineering specifications in the first place. In the current dispute it appears that some

pipe was defective. And the construction procedures did not afford the proper "standard of care."

4. It is not a good idea to let a pipe fabricator provide design calculations for the selection of either the appropriate pipe material or its characteristics. Even worse, never let a pipe supplier write engineering specifications for any pipeline project. The engineer's duty to her/his client is to start with the geotechnical data of the site in question and consider several pipe materials for consideration and design them all independently using national standards and fundamental engineering principles.
5. In major projects of some significance where significant cost savings are possible, if latest design technologies such as either the German ATV or the Finite Element Method could be used, every effort should be made to use such technologies to demonstrate to the owner the benefit of using such methods of design. Once reasonable designs, which could be defended in case of possible claims or litigation later on, then vendors who could supply such materials should be checked for their past records, before final decisions on pipe material specifications are made. Personal bias toward or against any of the pipe material should be completely avoided at all cost and the decision to include or reject a particular pipe material should be based purely on engineering merit.
6. The key assumption which will be made during the design is that the contractor will furnish pipes without flaws. Therefore, proper quality assurance/control program needs to be developed and implemented throughout the pipeline project to ensure that these design assumptions are met. This may involve placing well-qualified representatives of the owner in the shop of the pipe fabricator to observe all manufacturing and quality control tests performed to make sure that the pipe passes the necessary QA/QC measures. The author has been in numerous pipe plants to protect the interests of the owner on different continents and have seen some outrageous conduct during his professional life.
7. In most cases, what is given in the consensus standards such as those from ASTM may not prove to be adequate to protect the interests of the Owner; thus, it is the duty of the engineer of record to develop and implement additional engineering requirements to be met by the pipe fabricator to ensure good quality is used throughout on the project.
8. It is extremely important for the owner to ensure that a proper pipe material selection process is used to arrive at the final design considering the following attributes:
   - Availability
   - Corrosion Resistance

- Abrasion Resistance
- Hydraulic Roughness and Flow Capacity
- Structural Strength
- Structural Stiffness
- First Time Installed Cost
- Operation and Maintenance Cost
- Bedding Requirements and Options
- Handling and Shipping
- Repair of Damaged Pipe
- Ease of Future Connections and Expansion
- Types and Effectiveness of Joints
- Speed of Construction
- Track Record
- ASTM and Other Standards
- Design Methods and Quality Control
- Level of Inspection Required at Pipe Plant
- Level of Inspection Required During Construction
- Fittings
- Transitions
- Manholes

9. In summary, the pipe designed and specified by the City of Portland would have worked well for this project had the contractor followed proper procedures for dewatering, trench preparation, pipe laying, and bedding placement. The impact of the March 1993 earthquake in the area did not have an appreciable effect on the pipeline. Also, the subsequent road construction activity had no impact on the concrete pipe.
10. There are times when being engaged in lengthy litigation and expending additional resources on members of the legal profession and our court system is not the best way to settle our differences. It is so happened that a quick out of court settlement of this dispute in favor of City of Portland was possible due to the cooperation of all parties involved in this particular case.

CASE HISTORY OF TUNNEL CONSTRUCTION, LOWER NORTHWEST INTERCEPTOR PROGRAM

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ABSTRACT

The recently completed Lower Northwest Interceptor Program (LNWI) in Sacramento, California comprises, ten pairs of microtunnels totaling 14,417 ft, three horizontal directionally drilled bores totaling 3,154 ft, and two large diameter tunnels under the Sacramento River totaling 3,953 ft. This paper describes the problems encountered with each trenchless crossing, as well as lessons learned with respect to shaft construction, ground control at shafts during tunnel break-in and break-out, frac-out and heave, settlement, obstructions, coordination with permitting agencies, and application of geotechnical baseline reports.

INTRODUCTION

LNWI Program Description

The Lower Northwest Interceptor (LNWI) is a 19 mi long pipeline constructed for the Sacramento Regional County Sanitation District (SRCSD) that will convey sewage from the growing northern Sacramento County area and the City of West Sacramento to the Sacramento Regional Wastewater Treatment Plant in Elk Grove as shown in Figure 1.

The LNWI is comprised of ten projects including two new pump stations, five pipeline projects and 14 tunneled crossings. The projects were grouped into seven design and construction contracts, and five construction management contracts, all of which were coordinated by Montgomery Watson Harza (MWH) as Program Manager. Total Program cost was about $600 million.

Surface and Subsurface Conditions

The topography of the LNWI alignment is relatively flat except for canals and river levees, and railroad and highway embankments. Land use along the alignment ranges from new housing developments adjacent to a freeway interchange at the northern end, to rural farm land crossed by drainage and irrigation canals, to an urban environment where the alignment passes through the City of West Sacramento, to wetlands.

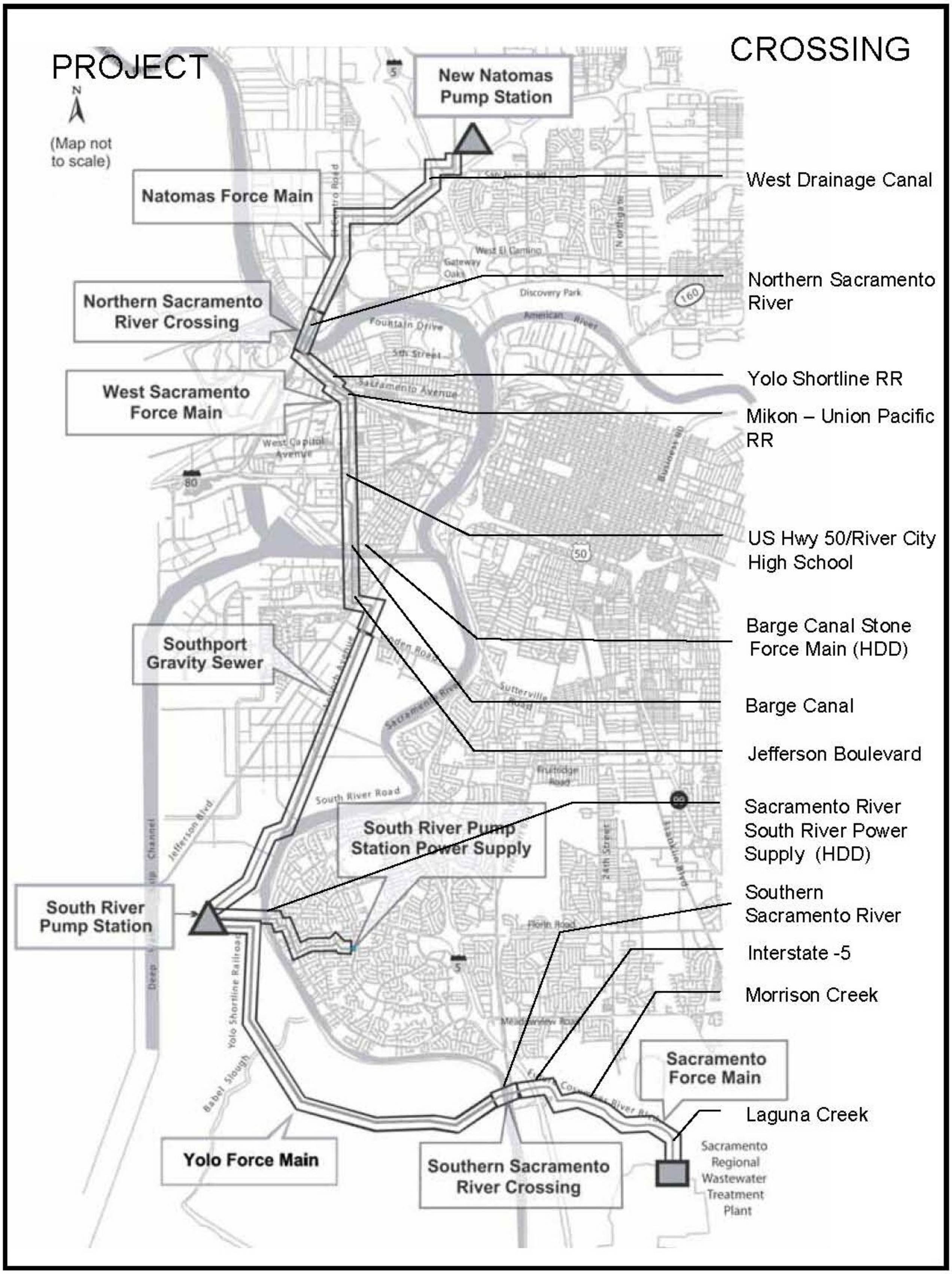

Figure 1 Lower Northwest Interceptor

Geological conditions along the alignment are characterized by Holocene age (less than 10,000 years old) fluvial and overbank deposits in a generally complex sequence of discontinuous lenses of inter-bedded clays, silts, sands and fine gravel. The subsurface profile is divided into shallow deposits comprised of active Sacramento River sediments (Qsc) and recent alluvium (Qa), underlain by Basin Deposits (Qb) generated by meandering and seasonal flooding of regional streams and rivers, and the older River Bank Formation (Qrl) formed by alluvial fan material emanating from the Sierra-Nevada Mountains to the east. (Nagel and Nonnweiler, 2003)

Tunneling conditions ranged from very stiff to hard clays and silts, sometimes exhibiting squeezing behavior, to loose to very dense silty sand, and medium dense to very dense gravel occasionally exhibiting flowing characteristics. Groundwater along the alignment is uniformly high, ranging from a depth of 10 ft during the dry summer months to ground surface during the wet winter season. Geotechnical Baseline Reports (GBR) were prepared for all LNWI contracts including open cut excavations as well as trenchless crossings.

PLANNING AND DESIGN CONSIDERATIONS

During preliminary planning and design the location and length of the trenchless crossings were determined considering a number of physical and environmental constraints that would otherwise preclude open cut trenching methods. These constraints included highways, railroads, and waterways. The feasibility of various alternative construction methods for the proposed trenchless segments were evaluated in order to develop preliminary cost estimates and to identify key design issues. The preliminary design is described in Nagel and Nonnweiler, 2003.

During final design some crossings, originally considered to be trenchless, were converted to open trench. The longest and most complicated tunnel crossings, the Northern and Southern Sacramento River Crossings, the Barge Canal, and the U.S. 50/River City High School Tunnel, were the subject of a number of alternative designs and risk assessment workshops.

CONTRACTING AND BIDDING

All LNWI construction contracts required that tunneling contractors/subcontractors be pre-qualified. Specifications for tunnel boring machines (TBM), microtunneling, horizontal directional drilling (HDD) equipment, and shaft excavation and shoring varied from contract to contract. All contracts contained requirements for contractors and tunneling subcontractors to participate in a formal partnering process. The contracts also required establishment of a Disputes Review Board (DRB).
Table 1 summarizes, for each LNWI crossing, project name, design consultant, construction manager, prime contractor and tunneling subcontractor, length of crossing, tunnel diameter, tunneling method and equipment, casing type and diameter where applicable, carrier pipe diameter and type, total cost of crossing excluding mobilization and incidental costs, and unit cost per linear foot of tunnel.

CONSTRUCTION

Following is a description of the problems experienced during construction of each trenchless crossing. Production data including: pipe or segment installation time, penetration rate, and tunneling efficiency are summarized in Table 1.

West Drainage Canal

No significant problems were encountered while driving the twin West Drainage Canal microtunnels and no settlement was recorded. However, arcs had to be cut in the waler flanges at the break-out and break-in locations in the launching and receiving shafts to accommodate the tunnel seals. In addition a diagonal strut interfered with placing casing pipe directly into the launching shaft and the receiving shaft was too small to accommodate the entire machine. Therefore the MTBM had to be disassembled before removal. (Doig et al, 2006)

Northern Sacramento River

Issues during construction of the Northern Sacramento River Crossing (NSRC) tunnel (Togan et al, 2007) included:

- A permit from the State Reclamation Board (SRB) prohibited any construction activity between the levees of the Sacramento River between 1 November and 15 April. To avoid schedule risk related to this blackout, the contract documents required the contractor to mobilize two TBMs. Following award, the contractor successfully proposed a Construction Incentive Change Proposal (CICP) that allowed him to mobilize a single TBM. This resulted in a net saving of $1,330,085 to the project. The contractor's revised scheduled called for mining the crossings sequentially during the Summer of 2005 starting with the Southern Sacramento River Crossing (SSRC) and completing the NSRC prior to the blackout period. However, delays experienced with the SSRC related to removing the TBM from the receiving shaft, caused a delay in mobilization for the NSRC. The revised schedule showed that tunneling activity would extend into the blackout period. Following several meetings and review of long range weather forecasts, the SRB agreed to allow the contractor to complete the tunnel and install carrier pipe into the blackout period.

- The TBM was stopped for a period of 5 d with the cutter head directly beneath an abandoned house belonging to the City of Sacramento at a depth of approximately 23 ft from tunnel crown to ground surface, in order to install the TBM's trailing gear. This stoppage resulted in loss of face pressure and settlement of 2 in. adjacent to the house which caused damage to the wooden structure. The contractor is responsible for either repairing or demolishing the house depending on the result of negotiations with the City of Sacramento.

TABLE 1 - LNWI TUNNEL SUMMARY

INTERCEPTOR		DESIGN	PRIME CONTRACTOR/	TUNNEL				CASING		CARRIER PIPE					EFFICIENCY	COST ($)*	
SEGMENT	CROSSING	CONSULTANT	TUNNELING SUBCONTRACTOR	LENGTH (LF)	DIA (IN)	METHOD	EQUIPMENT	OD (IN)	TYPE	ID (IN)	TYPE	AVG PIPE/SEG-MENT CYCLE TIME (Min)	BEST CYCLE TIME (Min)	AVG PENETRATION RATE (Ft/Min)	(TIME EXCAVATED/ TOTAL TIME)	TOTAL ($)	UNIT ($/LF)
Natomas Force Main	West Drainage Canal	Black & Veatch	Mountain Cascade/ Vadnais	380	82	Micro Tunnel/Pipe Jack	Soltau RVS 600A-S	78	Permalk Steel	Twin 60	Ameron 60-375 RCCP	188	56	0.27	39%	1,857,600	2,464
Northern Sacramento River Crossing	Sacramento River	Hatch Mott MacDonald	Affholder	1,975	198	EPBM	Lovat TBM	181	Pre-Cast Concrete Segments	Twin 60	Northwest Pipe C200 WSP	108	20	0.12	30%	11,658,391	5,894
West Sacramento Force Main	Yolo Shortline RR	CH2MHill	Mountain Cascade/Michaels	228	84	Micro Tunnel/Pipe Jack	Ackerman	78	Permalok Steel	Twin 60	Ameron C300 RCCP	30	70	0.50	30%	1,118,600	2,442
	Mikon - Union Pacific RR			394				78	Permalok Steel			30	70	0.45	30%	2,435,449	3,440
	Mikon - Union Pacific RR Inclined Risers							N/A				30	70	0.44	30%		
	U.S. Hwy 50/High School			1,402	84			78	Permalok Steel			24	60	0.68	55%	6,836,535	2,045
	Barge Canal			1,492				N/A				24	60	0.57	50%	6,330,895	2,178
	Barge Canal Inclined Risers		Mountain Cascade/Cherrington	313	36	Horizontal Directional Drill				Twin 24	HDPE	24	60	0.57	50%	656,000	400
	Barge Canal Stone Force Main		Mountain Cascade/Michaels	827	84	Micro Tunnel/Pipe Jack	Ackerman			Twin 60	Ameron C300 RCCP	N/A	N/A	N/A	N/A	2,424,545	2,582
	Jefferson Boulevard			313								120	90	0.40	24%		
South River Pump Station Power Supply	Sacramento River	HDR	The HDD Company	1,500	36	Horizontal Directional Drill		24 Steel		24	PVC	N/A	N/A	N/A	N/A	788,000	542
Yolo Force Main	Babel Slough	CDM	Las Vegas Paving/Vadnais	490	75	Micro Tunnel/Pipe Jack	Soltau RVS 600A-S	82.5	Permalok Steel	Twin 66	Ameron C300 RCCP	502	118	0.08	53%	2,684,475	5,535
Southern Sacramento River Crossing	Sacramento River	Hatch Mott MacDonald	Affholer	2,083	198	EPBM	Lovat TBM	181	Pre-Cast Concrete Segments	Twin 66	Northwest Pipe C200 WSP	160	25	0.10	25%	11,915,795	5,712
Sacramento Force Main	Interstate-5	URS	Rados/Fowler	640	83	Micro Tunnel/Pipe Jack	Soltau RVS 800	82.5	Permalok Steel	Twin 66	Ameron C200 WSP	536	170	0.07	66%	3,345,997	2,614
	Interstate-5 Inclined Risers			236				N/A			Ameron C300 RCCP					1,203,600	2,550
	Morrison Creek			880													
	Laguna Creek			600												8,077,988	2,729

* Cost includes: 1) shaft construction including ground treatment; 2) tunneling; and 3) lining or casing where applicable. Cost does not include: 1) mobilization/demobilization or other general costs; and carier pipe.

- Immediately after the earlier settlement and prior to tunneling beneath an occupied house, the TBM was stopped over a weekend with the cutter head directly under the north levee of the Sacramento River at a depth of 56 ft below the crest. The resulting dissipation of face pressure, and slow intermittent progress resulting from unavailability of computerized alignment information, caused observed surface settlement of up to 2 in.

Yolo Short Line Railroad

The Yolo Short Line Railroad (YSRR) twin, single pass microtunnels were driven without significant difficulties.

Mikon (Union Pacific Railroad)

The Mikon Crossing twin, double pass microtunnels and single pass inclined risers were driven without problems.

US Highway 50/River City High School

Significant issues that arose during construction of the twin U.S. Highway 50/River City High School (Hwy 50/RCHS) microtunnels included:

- The Hwy 50 crossing and RCHS were bid as two separate tunnels sharing the same launching shaft south of Hwy 50. The Contractor offered a CICP to microtunnel the entire stretch in a straight 1,400 ft alignment. These are among the longest microtunnels constructed in the United States to date.

- Because of the length of the bores, the tunneling subcontractor installed two intermediate jacking stations each separated by 600 ft, but they were not utilized.

- The microtunnels were launched from a shaft dewatered and shored with sheet piles just north of Hwy 50. Problems were encountered with flowing sand at the break-out points because of inadequate dewatering in mixed faced (clay over sand) conditions. This resulted in inflow of water and sand into the shaft, and settlement just outside the sheets between the shaft and the freeway. The shaft had to be filled with water on several occasions to equalize hydrostatic pressure in order to allow contact grouting from the surface to be performed.

- A significant technical problem to be overcome was that the portion of the tunnel under Hwy 50 was the first part of the crossing to be mined but also the part that had to have a steel casing. The contractor chose to microtunnel/pipe jack 60 in. RCCP pipe under the highway and the high school to a point 225 ft short of the receiving shaft. At that point the RCCP was fitted with a specially made flange that allowed connection to 72 in. ID Permalok steel casing. The microtunnels were then holed-through to the receiving shaft. The steel carrier pipe was then installed inside the casing pipe, welded and grouted into place.

- Ground heave and “frac-out” of slurry as well as bentonite grout occurred during mining of both drives under the high school track, basketball courts and tennis courts. These frac-outs occurred about 50 ft behind the cutter head and were thought to be caused by a combination of latent face pressure from the MTBM and grout pressure trapped under about 15 ft of clay cover. All fluid was cleaned up and heave repaired by the contractor.

Barge Canal WSFM

Problems encountered while constructing the twin single pass Barge Canal microtunnels included:

- Excavation of the launching shaft on the south side of the Barge Canal took about 1 yr longer than originally anticipated. Although the shaft was designed to be constructed in the wet with a mass concrete tremie seal in the invert, the Contractor chose to dewater the shaft and excavate in the dry. Problems were encountered with split sheets while dewatering and excavating the shaft for the first time even though the ground at the break-out points on either side of the shaft had been jet grouted. This led to a series of partially effective measures to control the inflow of water and soil over the next 12 months, and repeated flooding and unwatering of the shaft. These measures included additional jet grouting, construction of a concrete diaphragm wall adjacent to the sheet piles, contact grouting from the surface and from ports drilled into the sheet pile walls. After reaching the invert elevation a structural concrete slab was poured with relief holes, while the shaft continued to be dewatered. Because of concerns about the condition of the mixed face ground (sand and gravel over clay) at the break-out points in the shaft, the Contractor chose to install concrete collar seals poured inside steel forms with hinged steel doors at each tunnel eye, flood the shaft and cut out the sheets underwater with divers to guard against flow of cohesionless material into the shaft. Finally, because of the thin concrete invert slab, the designer became concerned with rebound of soil under the bottom of the shaft, and that subsequent re-consolidation during shaft backfilling would cause the carrier pipes to deflect beyond design tolerances. Therefore, the shaft was backfilled with light weight controlled density fill, produced with light weight aggregate, to reduce load on the foundation. According to the original schedule the Barge Canal Crossing was to be the second crossing to be microtunneled but ended up being the last. The contractor bore responsibility for most of the delays and cost involved, however change orders resulting from shaft excavation totaled $450,000 and 35 d of additional contract time (Mueller et al, 2006).

- Lower than anticipated rates of advance were achieved in both bores because of sticky clay conditions near the receiving shaft.

- A steel obstruction of unknown nature was encountered in the east bore approximately 130 ft short of the receiving shaft. The MTBM pushed through and over the obstruction with only minor damage to the outside of the forward shell. The obstruction caused a 1 d delay.

- Frac-out of slurry occurred approximately 150 ft short of the receiving shaft when there was about 20 ft of cover over the top of the MTBM.

Barge Canal Stone Force Main (HDD)

While constructing the twin HDD drives for the City of West Sacramento's Stone Force Main crossing of the Barge Canal, drilling fluid migrated horizontally a distance of about 30 ft to the Barge Canal microtunnel launching shaft. This occurred at a point near the southern end of the crossing when the drill string was at a depth of about 26 ft. A claim for a Differing Site Condition (DSC) for alleged gravel was settled for $10,000.

Jefferson Boulevard

Problems that occurred during construction of the twin, single pass Jefferson Boulevard microtunnels for the WSFM and the single microtunnel for the Stone Force Main included:

- The first microtunnel for the WSFM was the west bore under Jefferson Boulevard. Approximately 6 in. of roadway heave was experienced probably due to excessive face pressure in conjunction with relative shallow clay cover.
- During the second day of tunneling for the first bore, an unmarked low pressure, 4 in. diameter natural gas line owned by Pacific Gas and Electric (PG&E) was punctured by the cutter head. This resulted in closure of Jefferson Boulevard over a weekend while the pipeline was re-routed and the abandoned pipeline removed. SRCSD has submitted a claim to PG&E to cover additional contractor costs and the City of West Sacramento emergency response due to striking the unmarked gas-line. PG&E has submitted a counter claim against the contractor.
- Frac-out of slurry was observed at the south edge of Jefferson Boulevard near the receiving shaft for both drives. The frac-out probably occurred in the center of the roadway and was associated with the heave but slurry was trapped by the asphalt and only appeared at the roadway shoulder.

Sacramento River, South River Pump Station Power Supply (HDD)

A multi-chamber HDPE conduit pipe was pulled inside a steel casing installed under the Sacramento River by HDD as part of the Sacramento Municipal Utility District's (SMUD) power supply to the LNWI South River Pump Station. During grouting of the annular space inside the steel casing, one of the four conduit chambers partially filled with grout. After repeated attempts to ream out the blockage with no success, the contractor chose to install an additional conduit at his own expense.

Babel Slough

The first (west) microtunnel drive of the Babel Slough crossing encountered a nest of buried railroad ties that had been discarded in a previously unidentified dump over a distance of approximately 160 ft starting at a point about 40 ft from the launching shaft. The MTBM cut through the wood and completed the remainder of the tunnel without incident. The second bore did not encounter any obstructions. Approximately one day was lost due to slow advance rate while mining through the ties. A claim for DSC was settled for $10,500.

Southern Sacramento River

Problems during construction of the SSRC (Togan et al, 2007) tunnel involved:

- Bubbles were observed in the Sacramento River as the tunnel face passed underneath. As there was no evidence of frac-out of ground conditioner or grout from segment grouting, these bubbles appear to have been associated with EPB face pressure.

- On the west side of the river, as the TBM approached the receiving shaft at a depth of about 50 ft, the tunnel passed from relatively cohesive clay to sand. This resulted in a maximum of 4 in. of surface settlement before the TBM operator was able to make the necessary adjustment to face pressure. The settlement did not result in any damage as it occurred in an agricultural field, and further tunneling demonstrated classic ground control including minor heave associated with low cover.

- As the TBM approached the receiving shaft on the west side of the Sacramento River, probe holes were drilled into the 30 ft long jet grouted block of soil at the tunnel eye behind the sheet pile shaft wall. The contract mandated contact grouting of the jet grout block to control water inflow to specified requirements, was not fully complied with, nor was a formal seal, similar to the launching shaft seal, installed at the hole-through point as initially proposed by the contractor. A port with a guillotine gate was installed through the sheets to observe ground and water conditions. The shaft was initially dry, however as the TBM advanced toward the shaft, water inflow increased and was periodically arrested by contact grouting. Finally the bottom of the TBM punctured the sheet pile wall accompanied by an inflow of water and sand into the shaft. The sheets were cut out and the TBM pushed into the shaft. However, water and sand continued to flow into the shaft at the base of the TBM. The ground loss resulted in settlement of the ground surface outside the sheetpile shaft wall. In order to control ground loss, the shaft was flooded to equalize hydrostatic pressure. Disposal of dewatering flow from the shaft into the adjacent river was constrained by the permit from the Regional Water Quality Control Board. Therefore, an alternative solution was developed whereby ground water was pumped into temporary holding ponds and then into the dewatering pipe from the adjacent Yolo Force Main Project and conveyed to a dewatering treatment facility 5 mi away prior to discharge into the river. Because of these problems removal of the TBM, and consequently mobilization to the NSRC was delayed by three weeks. A settlement of $240,000 was reached with the contractor.

- The TBM's trailing gear became stressed as it navigated the 1,000 ft radius vertical curve between opposing 6% grades under the river. Bolts had to be replaced and welds strengthened in order to proceed.

Interstate-5

Issues during construction of the twin Interstate-5 microtunnels and inclined risers included (Doig et al, 2006):

- Hot-tap observation holes were drilled through the sheet pile shaft walls prior to cutting out to launch the MTBM. Water and soil flowed into the shaft through a small gap <1-in. between the sheets and the jet grouted block outside the shaft. Multiple stages of grouting with micro-fine cement were required to fill the gap and seal off flow. It was not clear whether the gap was the result of jet grout not effectively reaching the sheets or the sheets having deflected away from the jet grouted ground during shaft construction.

- There was a problem with the Permalok casing "egging" at some of the joints during mining. Efforts to identify the cause of the problem were unsuccessful, despite intervention by the pipe manufacturer. Ultimately, the egging was accepted as the pipe had sufficient clearance to allow installation of the carrier pipe.

- On the first run under I-5, the MTBM got stuck approximately 20 ft short of the receiving shaft. Efforts to free the machine were unsuccessful. The tunnel rose sharply along its length so that the receiving shaft was only 24 ft deep. As the tunnel was clear of the freeway by this time, and under a field, the machine was dug out and the pipe finished in open-cut. The machine head was seen to be heavily plugged, which could indicate that the slurry cleaning system had not been operating effectively.

Morrison Creek

Problems recorded during construction of the twin Morrison Creek microtunnels included:

- During the first (west) drive, the MTBM became stuck about 1.5 ft short of the receiving shaft. The reason for this was that the crew took Sunday off and when they returned to work they were unable to budge the machine. As with the I-5 drive, the head was plugged. After several days of trying to resume tunneling, the machine was excavated and removed. This unanticipated excavation occurred very near a protected wetland but was completed without incident.

- The MTBM rate of advance was significantly impacted by a controversy between the prime contractor and the tunneling subcontractor regarding removal of spoil from the slurry settlement tank. Because solids were not removed in a timely manner slurry denser than desired was re-circulated to the face which resulted in reduced penetration rates.

Laguna Creek

Mining production during construction of the twin, single pass Laguna Creek crossing was impacted because three drive motors had to be switched out over the course of the two microtunnel drives.

LESSONS LEARNED

Following is a summary of the lessons learned from the case histories of the LNWI trenchless crossings described above:

- Tunneling contractors/subcontractors and other specialty sub-contractors should be pre-qualified including experience of the contractor, qualifications of operators, and history of equipment.

- Ground improvement by jet grouting at tunnel eyes does not in itself guarantee success in controlling inflow of water and ground movement.

Specifications should call for a careful, step-by-step, systematic approach of probing and contact grouting of the jet grouted ground behind shaft shoring accompanied by other methods if necessary, prior to launching the TBM.

- Launching and receiving shafts should be dewatered to minimize the possibility of flowing ground during tunnel break-out and break-in.

- Formal seals or double sheets should be specified at all shaft break-in and break-out points to minimize the potential for inflow of water and soil.

- No problems with frac-out into waterways was experienced on any of the HDD crossings, contrary to fears of a number of the regulatory agencies.

- Frac-out can be experienced on microtunnels with shallow cover and face, and grouting pressures must be carefully controlled. Implementation of contingency plans proved crucial in controlling these spills.

- Shaft design and construction by the prime contractor should be carefully coordinated with the tunneling subcontractor. Contractors should ensure that subcontractor designed shafts take into account the dimensions of the TBM, pipe jacking equipment, and seals, as well as casing and carrier pipe.

- GBRs proved to be a useful tool in defining subsurface conditions for comparison with conditions encountered but were not utilized to resolve any formal disputes on any of the trenchless crossings. However, the inclusion of GBRs in all LNWI contract documents may have avoided disputes, as the definition of expected conditions was clear.

- When in sticky clay mining should continue.

- Coordination with permitting agencies should be transparent over the course of design and construction to facilitate modifications to permits if required by unforeseen conditions.

- TBM stoppage under sensitive features such as levees, roads, and buildings, should be minimized by specifying continuous operation.

- Contractors should not be allowed to construct shafts without an approved submittal.

- Use of a conveyor mucking system alleviated safety concerns while mining on 6% grades.

- Even though the specifications called for settlement monitoring and contractor corrective action depending on the amount, settlement usually occurred faster than it could be measured, and stopping the TBM to assess the situation usually resulted in exacerbating the problem.

- Public outreach issues due to vibratory sheet pile driving are significant. Noise mitigation for 24 h tunneling should be considered, including the use of new, quite generators.

- Construction risks can be managed by an owner by engaging designers, construction managers, and pre-qualified contractors experienced in underground construction, working as partners.

REFERENCES

- Nagel, G. and J. Nonnweiler. 2003. Preliminary Design of Tunnels for the Lower Northwest Interceptor, Sacramento, California, USA, Proceedings of the Rapid Excavation and Tunneling Conference, New Orleans, 2003, Society for Mining, Metallurgy, and Exploration, Inc. Littleton, CO

- Nonnweiler, J., J. Hawley, and D. Young. 2003. Design of Lower Northwest Interceptor Tunnel Beneath the Sacramento River, Proceedings of the Rapid Excavation and Tunneling Conference, New Orleans, 2003, Society for Mining, Metallurgy, and Exploration, Inc. Littleton, CO

- Mueller, C., and G. Klein. 2006. Managing Design Changes and Risk During Construction. Proceedings of the North American Tunneling 2006 Conference, Chicago, 2006,
- Doig, P., and A. Page. 2006. Microtunneling on the Lower Northwest Interceptor, Sacramento, California, Proceedings of the North American Tunneling 2006 Conference, Chicago, 2006,

- Togan, T., Martz, D,, Chen, W., Crow, M., Young, D., Moler, B., Norris, S. 2007. Construction of the Sacramento River Tunnels on the Lower Northwest Interceptor Sewer, Sacramento, CA, USA. Proceedings of the Rapid Excavation and Tunneling Conference, Toronto, Society for Mining, Metallurgy, and Exploration, Inc. Littleton, CO

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Integrated Leak Detection At Dallas Water Utilities

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Abstract

Dallas Water Utilities employs an innovative leak detection program which utilizes a variety of technologies to detect, locate, and repair leaks in both their water distribution and transmission systems. Over the past twelve months, this program has located over 200 leaks in Dallas' water distribution system, for an annual savings estimated at 100 million gallons of water. Over the past 2 years, Dallas has expanded their leak detection program to include the use of the Sahara™ leak location system on their large diameter transmission mains.

This paper discusses the technology and methodology used in Dallas' leak detection program, outlining the results achieved, and plans for further use of this program. Specific cases will be discussed in detail, with a focus on the innovative use of new technology.

Background

Dallas Water Utilities was founded in 1881 and is a not-for-profit department of the City of Dallas. Dallas Water Utilities provides water and wastewater services to about 2.4 million customers in Dallas and 26 nearby communities. The operations are funded solely by the water and wastewater rates paid by customers. The size of service area is about 699 square miles which is served by the water supply from five reservoirs and they are Lake Lewisville, Lake Grapevine, Lake Ray Hubbard, Lake Tawakoni, and Lake Ray Roberts. The water is treated at three treatment plants with a combined capacity of 900 million gallons per day. Dallas water utilities consist of 4,781 miles of water mains and 4,178 miles of waste water main.

Issue Identification

When water is under pressure in miles of buried pipes there will be water loss, all water systems face this reality. According to the American Society of Civil Engineers, "Every day 6 billion gallons of clean drinking water are lost due to old, leaky pipes and water mains" (Dinges, 2006). Sometimes it is easy to find the reasons for water loss, and other times it is very difficult. Aging infrastructure is also one of the reasons for water loss. According to an article in usnews.com, research indicates that there are 237,600 water mains break each year in the United States. One of the reasons to search for missing water is that by reducing water loss the operations team can improve the efficiency of the system, reduce operating costs for treating, pumping, storing water, and wear on the equipment. During normal conditions it is advisable to conserve water and prevent water loss; however, it becomes crucial during the drought conditions as North Texas is facing today.

Opportunity Identification

Dallas Water Utilities is taking a pro-active approach in conservation with a leak detection program. Team leaders identified “leak detection” as the first step in an aggressive conservation program. Dallas Water Utilities employs an innovative leak detection program which utilizes a variety of technologies to detect, locate, and repair leaks in both their water distribution and transmission systems. Over the past 12 months, this program has located over 200 leaks in Dallas Water distribution system, for an annual savings, estimated at 100 million gallons of water. Over the past two years, Dallas Water Utilities has expanded the leak detection program to include the use of the Sahara leak location system on large diameter transmission mains.

While large diameter transmission mains are considered less likely to suffer complete failures than small diameter pipes (Stone et al, 2001), they can nonetheless leak very frequently, and lose a tremendous amount of water. Thames Water reports that the average water loss from the over 400 leaks measured on large diameter mains (12 inches (300 mm) and larger) was roughly 40,000 gallons per day (150,000 liters per day), finding roughly 3.2 leaks per mile (2 leaks per kilometer) of transmission mains (Mergelas et al, 2006). These high leakage rates, combined with low failure rates, suggests that leaks in large diameter mains are slower to develop into complete failures of the pipe. If this is indeed the case, leaks in these large mains could be likely to go undetected for long periods of time.

The most common type of pipes in the Dallas Water distribution system are cast iron (CI), ductile iron (DI), and polyvinyl chloride (PVC) and reinforced concrete cylinder pipe (RCCP). Dallas Water distribution system consists of approximately 27,000 fire hydrants, 284,000 residential service connections, 5,000 commercial service connections, and 85,000 valves. The system is divided into 17 pressure planes with nine elevated tanks and 11 ground storage reservoir locations.

Technology

The Distribution Division conducted an investigation into which equipment to invest and use, which included surveying several municipalities, vendors, and suppliers. Equipment was selected on the basis of reliability, cost effectiveness and being the most advanced. The Distribution Division is using two types of correlators: AccuCorr 3000 Digital Correlators at a cost of $39,000 each, and Radcom SoundSens Logging Correlators at a cost of $19,500 each. Loggers, which are placed in district overnight and record the presence or absence of leaks (as opposed to correlators, which also locate leaks) are used as well; in particular, Patroller Units for Permalog Loggers at a cost of $8,000 each, and MK3 Permalog Loggers at a cost of $450 each. However, the Distribution Division continues to look at other technologies as they become available.

One new technology investigated is the Sahara Leak Detection system for large diameter transmission mains. Conventional external sonic leak detection equipment is ineffective in large diameter, 12 inches (300 mm) or greater, pipelines, as well as in non-metallic pipes, as the leak sound does not propagate far enough to be detected. Sahara overcomes this problem by inserting a tethered hydrophone inside the pipe, bringing the sensor to within one-half diameter of the sound. The Sahara system is effective in transmission mains 12 inches (300 mm) and greater, of all materials (Bond et al, 2004).

Methodology

Permalog MK3 Leak Noise Loggers are attached to each available pipe fitting over areas of the distribution network, and report the presence or absence of leaks in this area (the size of the area depends on the number of loggers used). AccuCorr 3000 Digital Correlator is used to pinpoint leak locations on various size and types of main. Radcom SoundSens Logging Correlator is used to pinpoint very small leaks or leaks that are in heavy traffic areas. This correlator can be deployed during regular business hours and programmed to record the data at night when the traffic is not as heavy or the usage is less. Correlators are known to be effective in small diameter pipelines, however leak signal strength dissipates rapidly within large diameter pipelines - pipelines over 12" (300mm) in diameter - inferring that the acoustic signal of the leak may not be practically detectable by correlators in large diameter pipe (Bond et al, 2004).

Dallas Water Distribution Division currently has two leak detection teams, one team of two people in the North Dallas area and another in the South Dallas area. The decision was made to enhance the operation, reduce travel time and improve response time by having one crew on either side of the city. The geographical division ensures that the distribution system is covered in a systematic manner. Management team is continuously looking to expand the program. The current goal is to survey the entire city in five years.

When the leak detection team locates and marks a leak for repair, the team will turn in a leak report identifying the location and type of leak such as main leak, fire hydrant leak or service leak. These reports are turned in daily to the Leak Detection Supervisor. This information is then entered into a database and a copy of the leak report is forwarded to the appropriate District Supervisor for repairs. After the repairs have been made the District Supervisor fills out the information on the leak report quantifying the water loss, type of leak such as joint leak, split pipe, or hole in pipe, name of foreman responsible for the repair , what date the repairs were made and how close the actual leak was to the mark made by the leak detection team. Man hours, equipment and material costs are captured on separate ticket.

Results

The Dallas Water Distribution Division has successfully surveyed 990 miles of mains, and identified 143 leaks through the survey, yielding an average of 0.14 leaks per mile (0.09 leaks per kilometer). They have also located and marked 406 known leaks for repair by the districts, saving over 130 million gallons of water since the inception of the program.

Dallas Water Utilities has contracted with The Pressure Pipe Inspection Company (PPIC) to survey a total of 13.36 miles of its large diameter transmission mains using the Sahara leak location system. These surveys identified 32 leaks, yielding an average of 2.40 leaks per mile (1.50 leaks per kilometer). These leakage rates for large diameter transmission mains, while slightly lower than the numbers found by Thames Water (Mergelas, 2006), confirm the high frequency of leaks on these large diameter transmission mains.

Dallas Water Utilities uses advanced technology such as Sahara leak detection to locate and pinpoint leaks on large mains, and remote field eddy current / transformer coupling testing to assess the condition of PCCP pipes and repair them before they fail.

Future Plans

Although there is a strong demand by repairs crews to have the leak detection crew personally aid them in pinpointing known leaks, DWU is committed to finding new leaks throughout their 4,772 mile water distribution and transmission systems using their internal systems and Sahara. To address the needs of the repair staff, Dallas Water Utilities will be adding two additional members to its leak detection team in October 2006. The enhancement will increase the amount of pipeline surveyed within the system as well as provide assistance to repair crews as needed.

Case Study: Deep Ellum / Fair Park

Dallas Water Utilities (DWU) provides water and wastewater services to about 1.9 million people in Dallas and 26 nearby communities and operates over 4,700 miles of water transmission and distribution pipelines.

Recently, the State of Texas and the Texas Administrative Code mandated a program of leak detection, repair, and water loss accounting for all water transmission, delivery, and distribution systems serving a population greater than 5,000. Accordingly, DWU designed a study comparing the various technologies used to detect and locate leaks within its water transmission system. According to Randall Payton, Senior Program Manager for DWU, "The goal of this study is to determine which technologies are effective at identifying and locating leaks in pipe of varying material composition and diameter. DWU will use the results of this study to minimize the cost of complying with the program while maximizing leak detection and location."

In order to positively identify the location of the leaks that would form the basis for this comparison, DWU contracted with the Pressure Pipe Inspection Company (PPIC) to perform leak detection surveys using the patented Sahara Leak Detection Technology.

Accordingly, in July 2005, PPIC inspected several small diameter pipelines in Fair Park using the Sahara system. The inspection was conducted in 17 different areas on pipelines constructed of 12 and 16 inch diameter cast iron (CI) and PVC. The overall inspected distance was 14,703 feet. Analysis of the data obtained during this inspection revealed a single small leak within the areas tested.

In August 2005, PPIC continued its assessment and inspected several small diameter water lines in Deep Ellum. This inspection was conducted throughout 31 different areas on pipelines constructed of 12, 16, 20, 30 & 36 inch diameter CI and/or PVC. The overall inspected distance was 35,229 feet. Analysis of the data obtained during this inspection revealed 17 leaks within the areas tested. These leaks ranged from small to large in magnitude.

Figure 1: Sahara Leak Detection

DWU staff review the inspection plan prior to conducting a Sahara leak detection survey on a 12” cast iron pipe in the heart of Dallas’s entertainment district, Deep Ellum.

Dallas Water Utilities verified the results of this inspection through excavation at GPS co-ordinates provided by PPIC, quoting a 100% accuracy record, and has repaired the distressed pipes.

Fair Park				
Diameter	**Type**	**# of Surveys**	**Distance Evaluated**	**Leaks Located**
12"	C.I.	8	8,034'	1
16"	C.I.	2	2,168'	0
12"	PVC	7	4,487'	0
Deep Ellum				
Diameter	**Type**	**# of Surveys**	**Distance Evaluated**	**Leaks Located**
12"	C.I.	18	17,089'	4
16"	C.I.	6	6,650'	3
20"	C.I.	2	1,542'	0
30"	C.I.	2	2,664'	3
36"	C.I.	2	2,195'	7
12"	PVC	1	1,591'	0

Conclusions

The goal of the program is to conserve water by minimizing water loss, reduce failures and improve service reliability and as a result save a precious resource, reduce the cost of production and save money. As explained by Dr. David E. Daniel, President of University of Texas at Dallas, “Extreme drought is inevitable. Do we appreciate the impact of unprecedented drought? Is there enough priority on public welfare? Is cost influencing our decisions? Is cost trumping safety?” These are the questions that the management team will have to ponder.

References

Daniel, D. E. (2006). *New Orleans Levee Failures.* Retrieved December 14, 2006 from presentation.

Dallas Water Utilities. (2006). *Interesting facts. Retrieved December 11, 2006* from http://204.66.40.189/dwu/water_interesting_facts.html

Dinges, C., (2006). “2005 Report Card For America’s Infrastructure.”, CCI/USACE Workshop, July 14, 2006

Lavelle, M. & Kurlantzick, J. (2002). *The Coming Water Crisis.* Retrieved December 11, 2006 from http://www.usnews.com/usnews/biztech/articles/020812/archive_022254.htm

Bond, A., Mergelas, B., Jones C., (2004.) “Pinpointing Leaks in Water Transmission Mains”, Proceedings of ASCE Pipelines 2004, San Diego, California, USA

Mergelas, B., Bond, A., Laven, K., (2006) “Financial Benefits from Seven Years of Water Loss Control utilizing the Sahara® System at Thames Water in the United Kingdom”, Proceedings of ASCE Pipelines 2006, Chicago, IL, USA

Stone, S., Dzuray, E. J., Meisegeier, D., Dahlborg, A., Erickson, M., (2001). “Decision-Support Tools for Predicting the Performance of Water Distribution and Wastewater Collection Systems.”, National Risk Management Research Laboratory, U.S. Environmental Protection Agency

Smith, J. B. (1989). *An Introduction to Water Loss and Leak Detection.* Retrieved December 12, 2006 from www.jbswater.com

Why All the Broken Pipe?

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Keywords

Transient analysis, pipeline failures, force main, pipeline investigation, and lift station

Abstract

The City of Wichita Falls (City) has experienced several problems with Lift Station No. 65 and the related force main since being placed in service. Several overflows called for the retrofit of a grinder pump that was installed upstream of the prison site in 1996. Electrical problems were also experienced, which may have contributed to later failures of the 10-inch force main. Despite constructing a new force main and installing pump control valves, the problems experienced by the Wichita Falls lift station and force main have not been alleviated.

The City consulted with Lockwood, Andrews & Newnam, Inc. to analyze the lift station and force main system and to address the issues they faced. Together with a local engineering firm, Cornerstone Engineering, the project team collected lift station and force main as-built drawings, O&M manuals, and other related system data. This data was utilized to build a computer model of the lift station and force main system for analysis. An analysis of the effects of transients on Lift Station Number 65 and the 10-inch PVC force main will be performed for three conditions:

1. Normal pump start-up,
2. Normal pump shutdown, and
3. Failure of all running pumps

Special attention was given to the size of the wet well and the frequency of operation of the pumps. Transient conditions will also consider failure of the pump control valves, and the minimum pressure criteria, based on information received from the force main manufacturer.

The force main failures will be reviewed with several issues in mind. The pipe material may be experiencing fatigue if a cyclic stress condition occurs at a high frequency and magnitude on a continuous basis over a period of time. The fact that the failure mode was the same in all cases in the 1994 line and the 2002 line suggests that the integrity of the pipe material was likely a cause of failure. In addition, the size of the line will be addressed.

Based on results of the transient analyses and data collected, the project team identified the cause of the force main failures and proposed the appropriate solutions.

Background

The City of Wichita Falls' (City) Lift Station Number 65 (Lift Station) and associated force main were constructed in 1994 to provide sanitary sewer collection service for the proposed James V. Allred Prison Unit of the Texas Department of Criminal Justice System that was first populated in 1995. Since being placed in service, the lift station has experienced numerous problems requiring certain equipment additions, modifications, and/or retrofitting of the original system.

One problem that continues to plague the Lift Station system is force main failures within the first 2,400 feet downstream of the lift station. These failures, which are almost exclusively joint-bell circumference cracks, began in 1997 and over time became more frequent. Between December 1997 and January 1999, the City repaired eleven joint-bell failures on the 10 inch SDR 26 PVC force main.

In an attempt to alleviate the failures, in 2000 the first 2,400 linear feet (LF) of the 10 inch force main were replaced with new pipe of the same class. In 2002, the Lift Station was fitted with 10 inch ball valves with adjustable closing rates, and the original APCO combination air valves were replaced with A.R.I. combination air valves. According to the City, the ball valves and combination air valves were intended to reduce the water hammer effect on the pipe joints. Nonetheless, beginning in 2002 the replaced section of the force main started experiencing joint-bell failures. Since that time, the City has recorded and repaired more than 26 force main failures within the same 2,400 LF section of force main.

Lockwood, Andrews & Newnam, Inc. (LAN), with assistance from Cornerstone Engineering, LLP (Cornerstone), was hired by the City of Wichita Falls to evaluate the Lift Station and its associated force main system and provide recommendations to alleviate these system failures. This report presents the findings and recommendations of this study.

System Information

Lift Station Number 65 is located along the north side of Kiel Lane between Huntington and Wellington Lanes. Construction of the Lift Station was completed in 1994, and the Lift Station was placed in service in 1995 when the Allred Prison began operation. The Prison was originally designed to be a 2,250 bed facility, and the Lift Station was sized accordingly for an average flow of 250,000 gallons-per-day (gpd) and a peak flow of 450 gallons-per-minute (gpm). However, expansion to the Prison has increased its capacity to almost 3,700 beds, presumably with a corresponding increase in flow to the lift station.

A 12 inch gravity sewer 6,450 feet in length delivers wastewater to the Lift Station from the Prison. The Lift Station then pumps the wastewater through approximately 20,000 LF of 10 inch force main eastward along Kiel Lane and ultimately into a 10 inch gravity sewer located on Airport Drive west of Interstate Highway 44. This 10 inch gravity sewer drains into the Lynwood West Addition sewer.

Assumptions and Criteria

Analyses of the affects of transients on Lift Station No. 65 and the associated 10 inch force main were performed for the following conditions:

1. Normal pump start-up,
2. Normal pump shutdown, and
3. Failure of all running pumps.

Transient modeling was performed using the Liquid Transient (LIQT) computer program. Transient modeling analyzes the possible occurrence and magnitude of a transient wave. Transient waves, also know as surges, occur in pipeline systems due to unsteady flow. A transient wave occurs as a sequence of pressure waves of varying magnitude that are greater than or less than the normal operating pressure of the force main. Effects of transients in pipelines can lead to the collapse and/or rupture of the pipeline dependent upon the nature of the transient wave and characteristics of the pipeline system.

The system model was based on as-built drawings and data gathered by Cornerstone regarding the existing pumps, control valves, and system operation. The calculated wave speed for 10 inch SDR-26 PVC was 1,138 feet per second. The wave speed was calculated assuming the pipe is rigidly anchored.

Each modeling condition was analyzed with and without the existing combination air valves being in service. Negative pressures are the typical result of pump failure scenarios. A positive pressure surge may also occur due to a pump failure scenarios when the water column returns to the pump and is suddenly stopped.. If the vacuum pressure falls to the vapor pressure of the liquid, the result is the creation of vapor pockets. The subsequent collapse of these vapor pockets may produce high pressure spikes and a high pressure transient wave propagating through the line further stressing the pipe. High surge pressures can also be a result of improper system operations.

Other assumptions, criteria, and conditions modeled include:

- Minimum allowable pressure criteria of -10 psi (10 psi vacuum).
- Maximum allowable pressure criteria of 160 psi (i.e. 10 inch SDR 26 PVC is pressure rated for 160 psi).
- System flows modeled represent maximum flow conditions.

Model Analysis and Results

The modeling analyses were performed utilizing the current lift station operations. The ball control valves were modeled with a one minute opening rate and a two minute closure rate. These opening and closure rates were observed during visits to the lift station. Field observations of the Lift Station operations, conducted on September 8, 2006, indicated the average cycle time is approximately 13 minutes. The Lift Station consists of two 900 gpm pumps that are operated in an alternating sequence. During the field visit, it was observed that pump one was in operation for approximately six and a half minutes and out of operation for approximately six and a half minutes before pump two cycled on.

In order to obtain current pressure and flow data for the Lift Station, the City monitored the system over a one week period, from September 14 to 22, 2006. The pressure and flow data were recorded just downstream of the Lift Station. The data was used to calibrate the transient model. The highest recorded pressure during the monitoring period was 82 psi.

The average daily flow recorded was approximately 700,000 gpd (486 gpm). Flow monitoring from previous flow recordings obtained in 2000 indicated a peak flow of approximately 1,300,000 gpd (903 gpm); therefore, to model a worst case scenario, a flow rate of 1,300,000 gpd (903 gpm) was assumed in the model.

Results of the transient model under the normal start-up and normal shutdown condition with the existing air valves in service resulted in a maximum pressure of approximately 80 psi, consistent with the observed system operation. The pressure near the lift station peaked to 80 psi during pump start-up. This pressure "spike" was attributed to the stagnant water column within the force main in between pump cycles. As the water column gained momentum the pressure stabilized and the maximum pressure experienced during the remainder of the simulation was approximately 76 psi. Similarly, the recently recorded pressure data indicates a maximum pressure of 82 psi during pump start-up. The pressure then decreases to approximately 70 psi for the remaining cycle, consistent with the modeling results. Neither the transient model nor pressure recordings indicated a surge pressure above 82 psi during the normal pump shutdown. A small transient wave was created during shutdown; however, the resulting pressures from the transient are less than the operating pressures.

The transient model results for the modeling of normal start-up and shutdown with the air valves out of service indicated a high pressure of approximately 80 psi. The air valves being out of service created a negative pressure condition within the force main. The model results indicated a negative pressure of approximately -14 psi (i.e. vapor pressure) could occur in the force main, if the air valves were out of service. While negative pressure of this magnitude may be cause for concern, the maximum pressure of 80 psi which resulted from this scenario indicates no cause for concern. This particular scenario does not appear to be the cause of the failures. The pipe was manufactured to withstand an operating pressure of 150 psi and a burst pressure of 500 psi. Additionally, based upon interviews with City personnel during the site visit, the air valves were described as being operational and only require occasional maintenance.

The final scenario modeled was failure of the operating pump. This scenario assumed that the ball control valves would experience a rapid closure of two seconds, based on the ball control valve project specification. Failure of the operating pump and subsequent rapid closure of the ball control valve did not create a transient wave greater than the pressure experienced during pump start-up.

Based on the transient analyses, the recurrent force main failures do not appear to be due to a transient problem. During this study, City personnel checked the air valves and verified that they were operational. The City checks the air valves on a monthly basis and cleans the valves on an as-needed basis. The current air valve spacing appears to adequately protect the system. The modeling results indicated that the transient wave that occurs in the force main did not produce high or low pressures substantial enough to cause damage to the force main. Graphs below show the pressures from the modeling results.

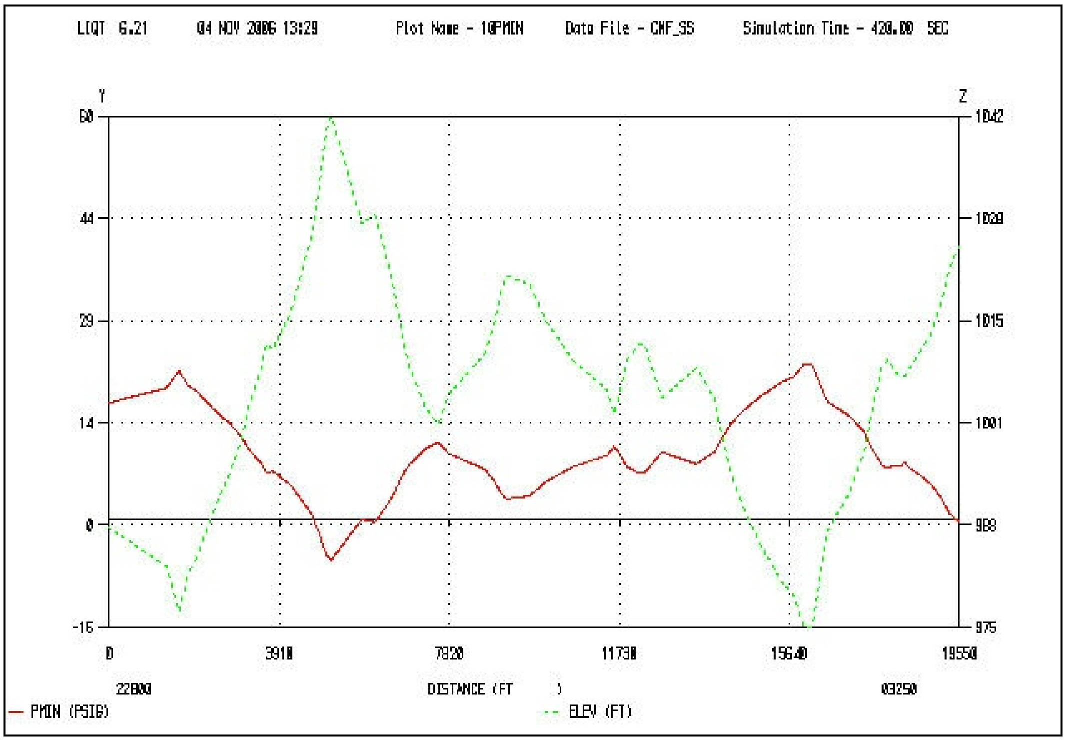

Figure 1: Force Main – Distance (ft.) vs. Minimum Pressure (psig) Under Normal Startup Operations with Surge Protection

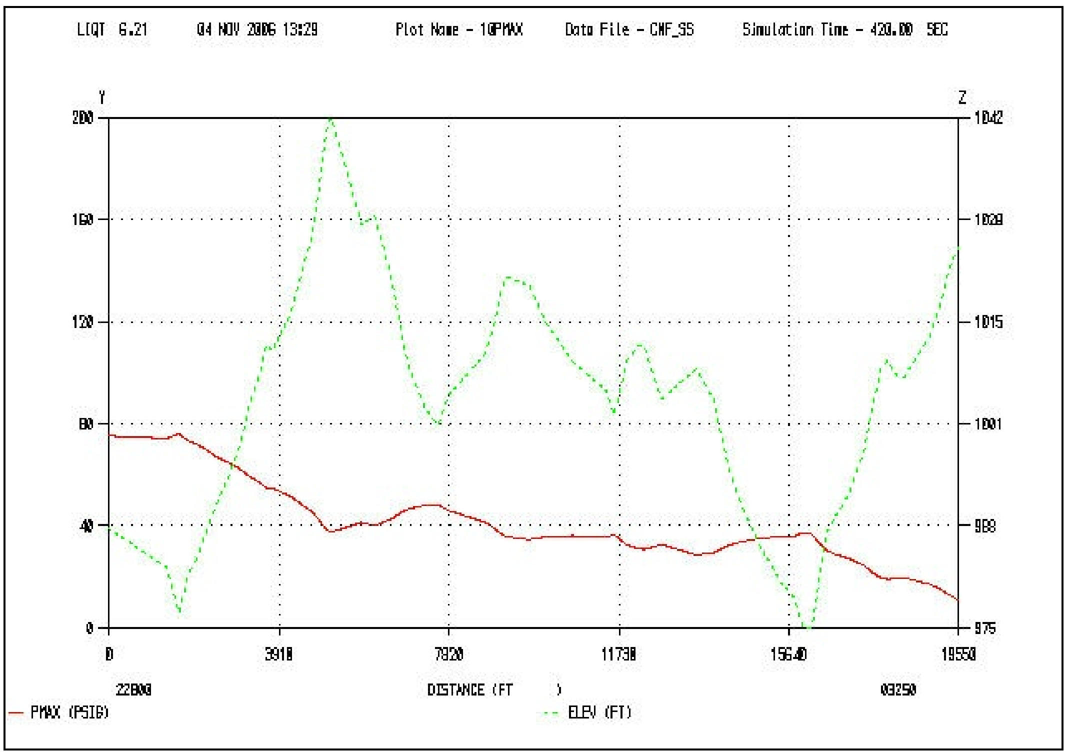

Figure 2: Force Main – Distance (ft.) vs. Maximum Pressure (psig) under Normal Startup Operation Conditions with Surge Protection

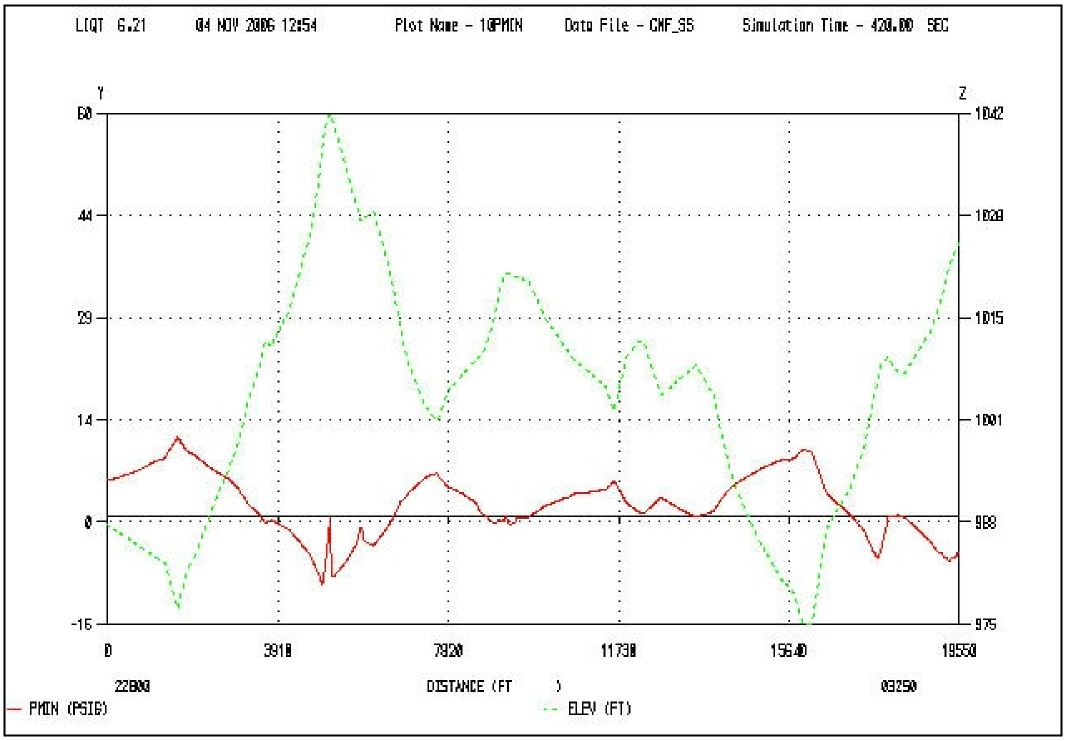

Figure 3: Force Main – Distance (ft.) vs. Minimum Pressure (psig) under Pump Failure with Surge Protection

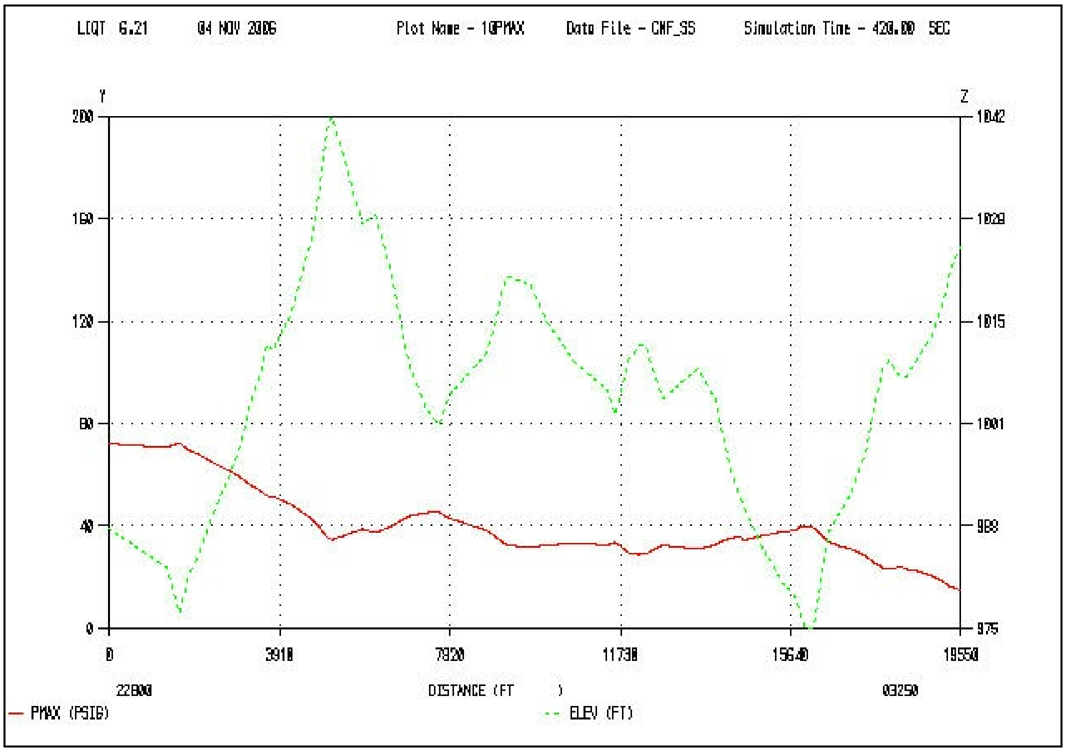

Figure 4: Force Main – Distance (ft.) vs. Maximum Pressure (psig) under Pump Failure with Surge Protection

Pipe Sample Analysis

Both the original and replacement force main pipe was constructed of 10-inch SDR 26 PVC manufactured by North American Pipe. The SDR 26 PVC pipe was manufactured to ASTM D2241 standards, which specifies a rated pressure of 160 psi and a burst pressure of 500 psi. Additionally, ASTM D2241 specifies that the pipe must have a minimum wall thickness of 0.413 inches.

A sample of the original 10 inch force main was obtained for analysis, since the findings of the transient analysis did not indicate the presence of a high pressure surge. Two samples of the original force main were taken just downstream from the Lift Station at Station 226+00 and Station 227+50. One of the samples had experienced a break around the circumference of the bell. The break appeared brittle and was directly in the gasket groove. The other sample had not experienced any failures. Both samples were first visually examined and then sectioned to allow further analysis. Photos of the force main that experienced the break are provided in Figures 5 and 6.

Measurements of the bell on the force main samples indicated the minimum thickness may not have been met in accordance with ASTM D2241. The thickness was reduced where the bell is expanded for gasket groove. A third sample was also obtained from the original force main to compare to the previous samples. The third sample also indicated that the minimum thickness may have not been in accordance with ASTM 2241. However, it is recommended that the services of an independent testing lab be acquired to analyze the force main, dimensions. The measurements in Table 1 were taken at the bell where the force main has been experiencing failures.

Table 1: Original Force Main Bell Dimensions

Force Main Sample 1	0.360 in.
Force Main Sample 2	0.379 in.
Force Main Sample 3	0.337 in.

Figure 5: Pipe Samples from Original Force Main

Figure 6: Pipe Samples from Original Force Main

During limited study of the pipe samples and based on field visits, no construction deficiencies were identified. Due to the nature and the consistency of the failures it was initially thought that the pipe was "over-stabbed" or "over-belled" during the construction, which would have been a potential cause of the failures. However, based on a visual inspection of the pipe, there is no evidence that the pipe was "over-stabbed" or "over-belled."

In regards to the force main failures, there is no evidence that the force main was not properly bedded. If the force main was not properly bedded, the transient wave observed in the transient modeling results and in the recorded pressure data may cause the force main to "flex" underground, resulting in possible pipe fatigue. If this were a problem, the frequency of the pump cycles, discussed below, would contribute to this type of failure.

Uni-Bell Pipe Association and North American Pipe Corporation were both contacted in regards to the force main failures. The representative from Uni-Bell Pipe Association was not familiar with the type of force main failure that is being experienced at the Lift Station. The representative from North American Pipe Corporation was only familiar with one other case in which PVC pipe experienced failures around the circumference of the bell. These breaks occurred in 2 to 3 inch diameter PVC piping that was being used for irrigation. According to the representative it was believed that the pipe failures were occurring due to the irrigation pipe flexing.

Wet Well Analysis

The wet well was originally sized for a flow of 250,000 gpd (174 gpm). The current average flow is approximately 700,000 gpd (486 gpm). The increased flow has caused an increase in pump cycles. The Lift Station's two 88 hp pumps should not individually have more than one start in any 15 to 20 minute period. Currently each pump starts once approximately every 26 minutes. There are approximately five total pumping cycles per hour (i.e., each pump cycling two to three times per hour). If the flow into the Lift Station continues to increase, the number of pump starts per hour will increase proportionally. If one of the pumps starts more than once during any 15 to 20 minute period, then the size of the wet well should be re-evaluated at that time.

Findings and Conclusions

Based on results of the transient modeling, review of the pressure and flow data from the City, and preliminary inspection of the original force main segments, the following findings were noted:

1. The transient analyses performed to date do not identify the problem related to a transient issue. The current lift station operations do not appear to be causing the force main failures. The transient model results, with and without air valves on the system, did not indicate high or low pressure surges significant enough to damage the force main. The current air valve spacing also appears to adequately protect the system.

2. Although the current pump cycle times are on the high side, they appear to be within an acceptable range for the type of equipment installed. An increase in wet well capacity would reduce pump cycles per hour, which will increase the useful life of the pumps.

3. From our limited study of the force main and from the site visits, no construction deficiencies were apparent.

4. Limited measurements of the bell on the force main samples indicate the minimum thickness specified in the ASTM Standards may not have been met by the manufacturer. The thickness of the pipe wall at the bell may be a contributing cause to the failures.

Given the large number of failures in this line, it is important to note that all the occurred in the same location. The failures also appear to be brittle, not ductile. PVC pipe is susceptible to fatigue failure when placed in a pressurized system with cyclic loading. This pertains to both irrigation systems where valve closures cause cyclic loading (and transients), as well as force mains. It was suspected that the force main was failing due to fatigue from cyclic loading. Another suggested possibility is the effect of hydrocarbons on a rubber ring gasket. Information from an outside source noted that hydrocarbons, such as gasoline, can cause a rubber ring gasket to enlarge as much as two or three times it original size. If the groundwater or soil where the force main is contaminated with hydrocarbons, the gasket could enlarge causing high hoop stresses in the groove area on the bell. The latter suspicion has not yet been investigated further.

References

ASTM International. ASTM Standard D 2241 – 00. 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA 19428-2959.

Shumard, PE., Dennis. North American Pipe Corporation. 2801 Post Oak Rd., Suite 600. Houston, Texas 77056.

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Determination of Pipe Pullback load for Horizontal Directional Drilling (HDD) Crossings by Finite Element Method

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Abstract:

This paper put forward a new method to determine the tensile loading during the installation of steel pipe using the horizontal directional drilling (HDD). There are several methods being currently used. Advantage and disadvantages of those methods are discussed. The paper describes a commonly used Finite Element technique along with sophisticated material model, which deals with softening and failure of geomaterials. It has been shown that using different meshes for different stages of progresses in installation by pulling the pipe through the bore, a reasonable variation of pulling force can be obtained along the length of installation. This method can be readily applied to any site by knowing its geotechnical characterization. The method of tensile force calculation along the pipe installation is initially developed in two dimensional geometric model, but it can readily be extended to three dimensional case as there is no complex geometric interaction between the pipe and the soil. In all stages of calculation, a new mesh is used with advanced stage of installation of pipe. The interface behavior of pipe and soil is modeled by a thin layer of weak soil constituting the smear zone. The distribution of pulling force along the line of installation is determined from the interpolation of mobilized displacements of each stage of pipe setting.

Introduction:

Different trenchless methods are extensively used in Europe and North America for installation of new pipes and for in-place rehabilitation of existing pipeline systems. The advent of the trenchless methods enabled the constructions of essential underground pipes, while simultaneously avoiding environmental hazard and costly disruptions of urban life and traffic, which results from open cut methods of excavation. Many developing countries like Bangladesh also foresee the use of the trenchless methods for their underground infrastructures in cities with very high densities of urban population. Dhar et al (2006) made a qualitative assessment of the relevance and applicability of various trenchless methods for construction and maintenance of underground utilities in the city of Dhaka in Bangladesh and revealed that each of the trenchless method has a great potential of using in most of the areas of Dhaka city. However, the rapid growth in the application and general acceptance of the methods were largely based on experiences published in various literature rather than on understanding of the mechanism from an engineering perspective. The need for understanding of the mechanism of load transfer to the pipe during and after installation is very important for widespread acceptability of the methods, particularly for long installation of large diameter pipes that may involve huge investment. This paper focuses on the investigation of load transfer on pipe during pull back operation when installation using Horizontal Directional Drilling (HDD).

Horizontal Directional Drilling (HDD) is a method for installation of new pipes that begins with boring a small hole, known as pilot hole, with a continuous string of steel drill rod. When the bore head and rod emerge on the other side of the pilot hole, a back reamer is attached with the rod and pulled back through the hole. The reamer bores out the pilot hole to facilitate pulling of the pipe connected to it through the hole. Pulling force develop on the pipe during pull back operation is investigated in this research using finite element analysis.

Existing Design Models for Estimating Pulling Forces:

Estimation of pulling force is required during design of HDD pipe installation in order to determine the size of the drill rig to be used and to set the maximum pulling force such that the pipe is not damaged during pullback operation. The capacity of drill rig required for installation of a pipe depends on the pulling force necessary to pullback the pipe. The pulling force is assumed to be contributed by the frictional drags due to friction between the pipe segment inserted and borehole wall and due to friction between the pipe section remaining at the surface and the ground surface, the hydrokinetic drag, the capstan effect force resulting from increased bearing pressure of pipe being pulled around a inside curve of a bend. Figure 1 shows the locations along the length of the pipe of the four parameters contributing to the pulling force.

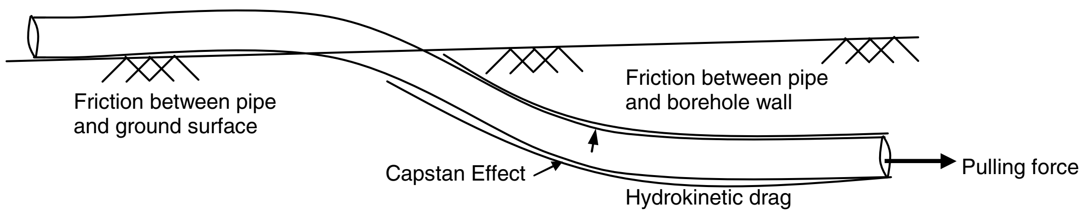

Figure 1. Parameters contributing to pulling force

Baumert and Allouche (2002) reviewed three design procedures used in North America for prediction of pulling forced during horizontal directional drilling, namely, Driscopipe Procedure, Drillpath Software, and PRCI Design Procedure. The Driscopipe procedure (Driscopipe, 1993) treats the lengthwise pipe profile as a series of straight-line segments linked together. The pulling force is calculated as the advancement of the pipe as:

$$T = \sum_{1}^{i} W_s L(\mu \cos\theta \pm \sin\theta) \tag{1}$$

Where, I = number of individual segments along the alignment
Ws = net weight effect of the pipe, pipe contents and buoyancy (kN/m)
L = Length of segment (m)
μ = friction coefficient for a section
θ = angle of inclination of the section with the horizontal (degrees)

Thus, the model accounts for the friction between the pipe and ground surface/ borehole wall considering the normal force from the net weight effect of the pipe, but neglect the pipe bending and hydrokinetic drag in calculating the pulling force (Baumert and Allouche, 2002). It was stated that the effect of hydrokinetic drag (mud drag) is minimal and can be neglected in load calculations.

Drillpath (1996) is a computer software that was developed to calculate the pulling force using the theory similar to the Driscopipe but employs stepwise segmental approaches to account for the effects of borehole curvature. The pipe segments are assumed to be capable of transmitting tension and compression through the joints and each segment is analyzed. Drillpath also neglects the effect of pipe bending (capstan effect) and hydrokinetic drag in the calculation.

PRCI (Pipeline Research Council International) also uses the segmental procedure to calculate the pull head force required to install a pipeline section (Huey et al., 1996). The pipe is divided into straight and curved sections as dictated by the borehole profile. The surface friction coefficient and borehole friction coefficient are used to consider the effects of frictional drag as in the two other methods. A constant fluid drag factor (i.e. 345 kPa) is used to account for the hydrokinetic drag in this method based on a research on a steel pipe dragged through a clean bentonite mud in Netherlands (Huey et al., 1996). However, the fluid drag was assumed negligible in the other methods (i.e. Driscopipe, 1993; Drillpath, 1996). PRCI method includes the capstan effect (the effect of bending stiffness of the pipe) through idealization of the pipe segments in the curved sections as simply supported beams with three points of contact. Solution of elastic beam deflection (Roark, 1965) was then used to determine the normal contact forces at the mid contact point, for a deflected shape matching with the shape of the borehole. A similar method is proposed in Lashen and Polak (2001) for calculation of pulling load based on the normal reactions at the contact points between the pipeline and the borehole walls.

Baumert and Allouche (2002) evaluated the performance of each of the three methods discussed above using two full scale tests where rig loads were measured with the advancement of pipe through two boreholes. Significant inconsistencies between the prediction and measurement were revealed in terms of both maximum pulling load and pull load values during advancement of pipes through the boreholes, indicating that the models failed to replicate the physical reality in the boreholes. Fundamental research is therefore warranted to develop a better understanding of the mechanism of load transfer during HDD installation. A finite element modeling of the installation process would contribute on the theoretical development of the mechanism and on determination of relevant parameters contributing to the installation process.

Finite Element Modelling:

Finite element analysis was used to model the interaction of the pulled out pipe and the surrounding soil during pull back operation. Pulled out pipe was modeled as truss element and the surroundings soil was modeled as elasto-plastic continuum element. Truss element does not consider the bending stiffness, neglecting the effects of bending stiffness of the pipe in the analysis. Thus, the contribution of the friction between the pipe and the soil will be revealed from the analysis. Pipes with three different mechanism of kink as shown in Figures 2, 3, and 4 have been considered to develop an understanding about the effects of kinks on the pulling force. Full length of the pipe was modeled to simulate the maximum rig load required to pull back the pipe.

The four noded quadrilateral elements with one-point integration have been used for modeling of the surrounding soil. The development of zero-energy mode due to the use of one-point integration is prevented by using Enhanced Assumed Strain (Reese et al., 2003) technology. The incremental elasto-plastic equation is integrated by explicit return mapping algorithm (Ortiz and Simo, 1986). Implicit dynamic relaxation (Siddiquee, 2006) method is used to solve the highly nonlinear systems of equations in a reasonably short time. Interface between the pipe and soil was also modeled using elasto-plastic continuum elements. However, the soil parameters were used as one-fourth of those used for surrounding soil, reducing strength and stiffness of the interface material. It was depicted that using of weak soil as interface material works reasonably well (Kotake et al., 1999) for large deformation problems. In this case updated Langrangian method of analysis is carried out in order to capture the large slip phenomena around the pipe. Figures 2 to 5 show the finite element meshes used for this study.

Elasto-plastic model of soil:

Elastic part:

The basic formulation of cross anisotropy can be modified with one basic natural assumption that the modulus of elasticity in each direction depends exponentially on the stress level of that direction (Hoque and Tatsuoka, 1998). That is given as follows,

$$\begin{aligned} E_v &= E_1 \sigma_v^m \\ E_h &= E_1 \sigma_h^m \end{aligned} \tag{2}$$

where E_1= "E_v or E_h at σ_v or σ_h=1.0" .

E_v and E_h are isotropic ($E_v = E_h = E_p$), when $\sigma_v=\sigma_h$, as supported by the experimental data (Tatsuoka et al.,, 1993). The following relation can be obtained from the previous Equation,

$$\frac{\nu_{hv}}{\nu_{vh}} = \frac{E_h}{E_v} = \left(\frac{\sigma_h}{\sigma_v}\right)^m \tag{3}$$

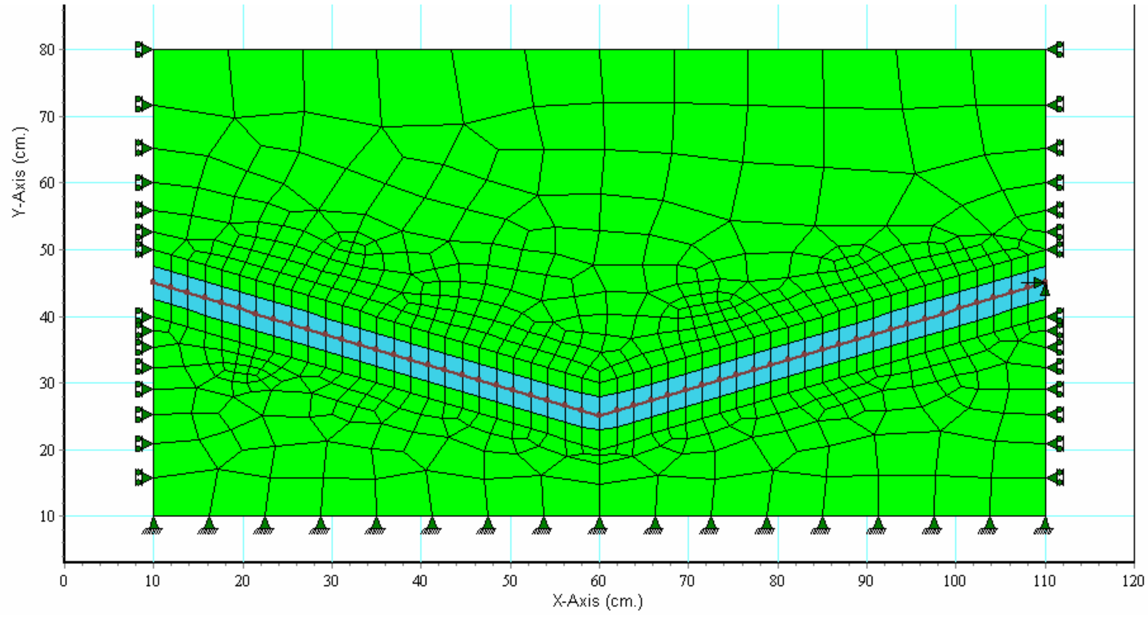

Figure 2 Mesh -1 of the one kink pipe layout.

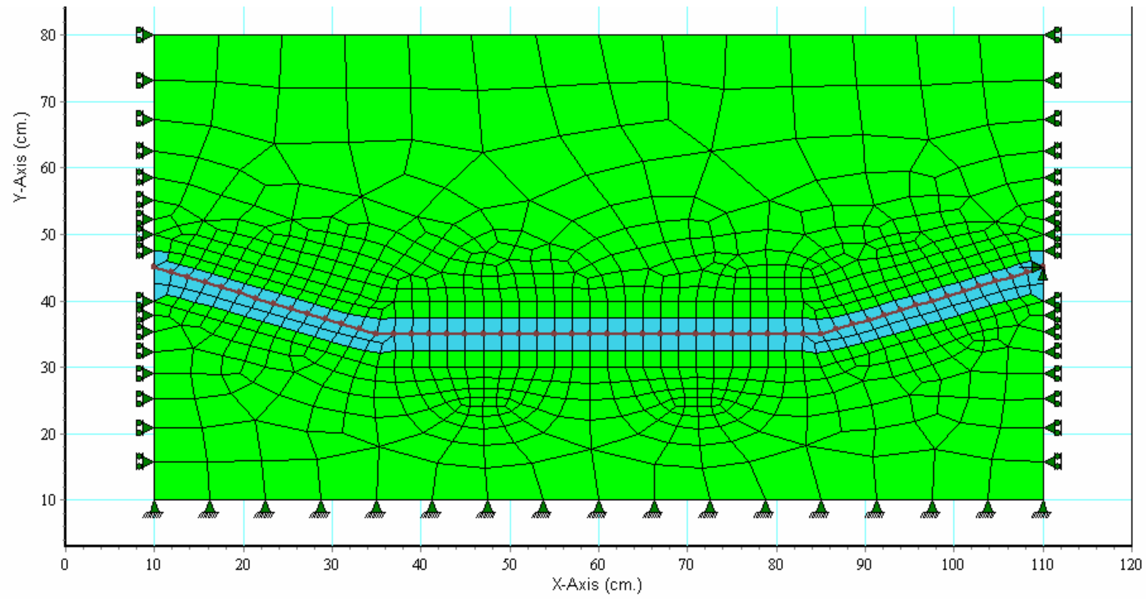

Figure 3 Mesh -2 of the two kink pipe layout.

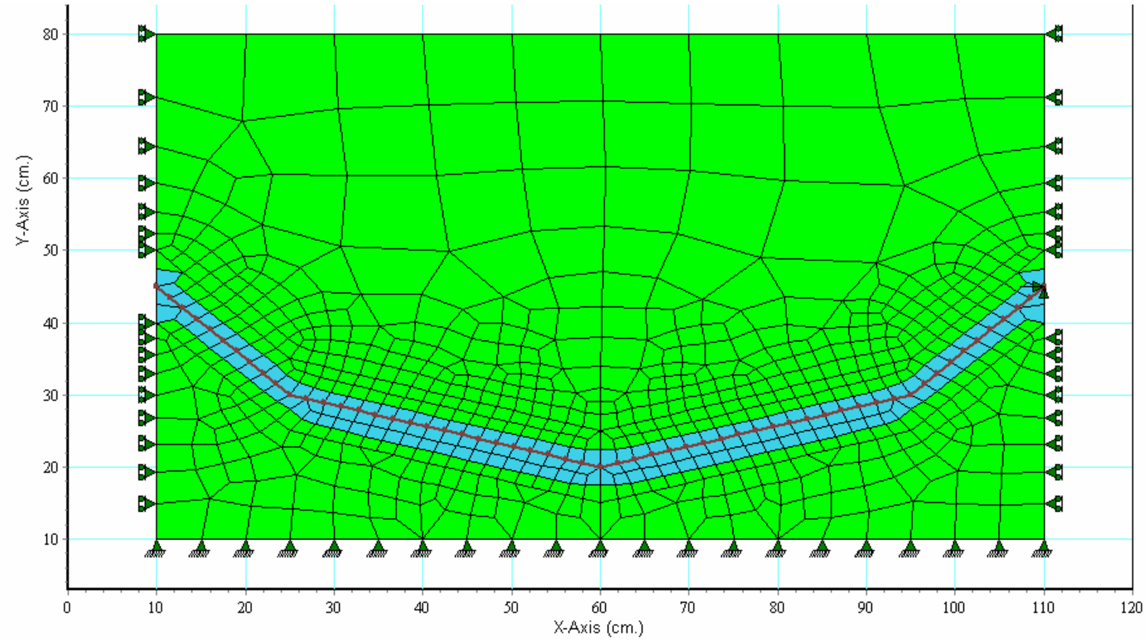

Figure 4 Mesh -3 of the three kink pipe layout

By using $\nu_{hv} = \nu_{vh} = \nu_0$ when $\sigma_v = \sigma_h$ (here, ν_0 is the isotropic Poisson's ratio), it can be assumed that,

$$\nu_{hv} = \nu_0 \left(\frac{\sigma_h}{\sigma_v} \right)^{m/2}$$
$$\nu_{vh} = \nu_0 \left(\frac{\sigma_v}{\sigma_h} \right)^{m/2} \tag{4}$$

The physical meaning of Eq. (4) can be interpreted by substituting $\nu_{hv} = -\delta\, \varepsilon_h / \delta\, \varepsilon_v$ in Eq. (4) and divided into the following two proportionality,

$$\delta\, \varepsilon_h \propto \frac{1}{\sigma_h^{m/2}}$$
$$\delta\, \varepsilon_v \propto \frac{1}{\sigma_v^{m/2}} \tag{5}$$

This means that $\delta\, \varepsilon_h$ increases as σ_h decreases, which is very natural to happen. The the anisotropic shear modulus, G_{hv} can be linked to σ_v and σ_h as follows,

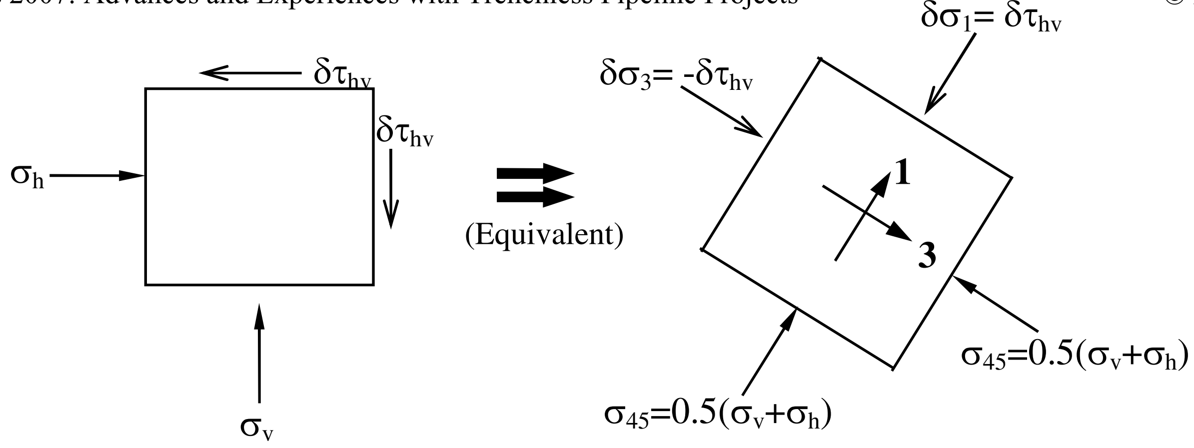

Figure 5: Stress transformation for principle stresses

From the above figure, the stress-strain relation for the principal stress direction can be written as,

$$\begin{bmatrix} \delta\varepsilon_1 \\ \delta\varepsilon_3 \end{bmatrix} = \begin{bmatrix} 1/E_{45} & -\nu_0/E_{45} \\ -\nu_0/E_{45} & 1/E_{45} \end{bmatrix} \begin{bmatrix} \delta\sigma_1 \\ \delta\sigma_3 \end{bmatrix} \tag{6}$$

Here, ν_0 is the isotropic parameter, since σ_{45} is the same in major and minor principal stress (1 and 3) directions. So from the above equation,

$$\begin{aligned} \delta\varepsilon_1 &= \frac{1}{E_{45}}\left(\delta\sigma_1 - \nu_0\delta\sigma_3\right) = \frac{(1+\nu_0)}{E_{45}}\delta\tau_{hv} \\ \delta\varepsilon_3 &= \frac{1}{E_{45}}\left(\delta\sigma_3 - \nu_0\delta\sigma_1\right) = -\frac{(1+\nu_0)}{E_{45}}\delta\tau_{hv} \end{aligned} \tag{7}$$

Hence,

$$\delta\varepsilon_1 - \delta\varepsilon_3 = \delta\gamma_{hv} = \frac{2(1+\nu_0)}{E_{45}}\delta\tau_{hv} \tag{8}$$

Now, following the same assumption as of Eq. (4),

$$\begin{aligned} E_{45} &= E_1\sigma_{45}^m \\ E_{45} &= E_1\left(\frac{\sigma_v+\sigma_h}{2}\right)^m \end{aligned} \tag{9}$$

Hence,

$$\begin{aligned} G_{hv} &= \frac{\delta\tau_{hv}}{\delta\gamma_{hv}} = \frac{E_{45}}{2(1+\nu_0)} = \frac{E_1}{2(1+\nu_0)}\left(\frac{\sigma_v+\sigma_h}{2}\right)^m \\ \Rightarrow G_{hv} &= G_1\left(\frac{\sigma_v+\sigma_h}{2}\right)^m \end{aligned} \tag{10}$$

Now further simplification can be attained by introducing $\alpha = \left(\sigma_h/\sigma_v\right)^{m/2}$ in Eq. (2), (4) and (10) and the general formulation becomes.

$$E_v = E_1 \sigma_v^m$$
$$E_h = E_v \sigma^2$$
$$\nu_{hv} = \nu_0 \left(\frac{\sigma_h}{\sigma_v} \right)^{m/2} = \nu_0 \alpha \tag{11}$$
$$\nu_{vh} = \nu_0 \left(\frac{\sigma_v}{\sigma_h} \right)^{m/2} = \nu_0 / \alpha$$
$$\begin{aligned} G_{hv} &= \frac{E_1}{2(1+\nu_0)} \left(\frac{\sigma_v + \sigma_h}{2} \right)^m \\ &= \frac{E_1}{2(1+\nu_0)} \left(\frac{1+\alpha^{2/m}}{2} \right)^m \end{aligned} \tag{12}$$

Plastic part:

The generalized elasto-plastic isotropic strain-hardening and softening model takes into account strain localization associated with shear banding by introducing a characteristic width of shear band in the additive elasto-plastic decomposition of strain. Unlike the method used by Pietruszczak and Mroz (1981), the direction of shear band is not specified in each element, but it is implicitly assumed that, in a given boundary value problem, the direction of shear band coincides with the global direction of local maximum shear strain among adjacent multiple elements. A generalized hyperbolic equation (GHE) (Tatsuoka et al., 1993) is used as the growth function of the yield surface of the generalized Mohr-Coulomb type, given by:

$$\Phi = -\eta I_1 + \frac{1}{g(\theta)} \sqrt{J_2} - K \tag{13}$$

where I_1 is the first stress invariant (i.e., hydrostatic stress component, positive in compression); and J_2 is the second stress invariant (i.e., deviatoric stress). The growth function is explained in details in Siddiquee et al. (1999, 2001a & b). The function $g(\theta)$ in Eq. 13 is the Lode angle function, defined as;

$$g(\theta) = \frac{3 - \sin \phi_{mob}}{2\sqrt{3} \cos \theta - 2 \sin \theta \sin \phi_{mob}} \tag{14}$$

In Eq. 13, η is the deviatoric stress at θ=30$^{\mathrm{o}}$ (on the π-plane), which is related to the mobilized angle of friction, φ_{mob}, as;

$$\eta = \frac{2 \sin \phi_{mob}}{\sqrt{3}(3 - \sin \phi_{mob})} \tag{15}$$

The plastic potential is defined as;

$$\Psi = -\alpha' I_1 + \sqrt{J_2} - K = 0 \tag{16}$$

This plastic potential function, of the Drucker-Prager type, is similar to the yield function except that $g(\theta)$ in Eq. 13 is equal to unity. Eq. 16 is employed so as to have differentiability at all stress states. The factor α' depends on the type of analysis. The factor α' used in the present analysis under plane strain conditions is linked to the mobilized dilatancy angle ν as;

$$\alpha' = \frac{\tan \nu}{\sqrt{9 + 12 \tan^2 \nu}} \tag{16}$$

$$\nu = \arcsin\left(-\frac{d\varepsilon_1^{ir} + d\varepsilon_3^{ir}}{d\varepsilon_1^{ir} - d\varepsilon_3^{ir}}\right) \tag{17}$$

where $d\varepsilon_1^{ir}$ and $d\varepsilon_3^{ir}$ are the major and minor irreversible principal strain increments (positive in compression), which are linked to each other through the Rowe's stress-dilatancy relation (Rowe, 1962);

$$\frac{\sigma_1}{\sigma_3} = -K\left(\frac{d\varepsilon_1^{ir}}{d\varepsilon_3^{ir}}\right) \tag{18}$$

where K is a material constant (equal to 3.5 for Toyoura sand in the present case). As the model has the yield function and plastic potential surface having different forms, it is one of the non-normal plasticity models or non-associated flow models (Vermeer and de Borst, 1984). Tables 1 and 2 show elastic and plastic parameters, respectively, used for the soil.

Table 1 Anisotropic elastic properties of sand

Material parameter	Values
Young's modulus, E0	1800
Ev/Eh	1.5
Exponent, n	0.41
Nu	0.17
Void ratio	0.66
Density (kgf/cm3)	0.00167
K0	0.33

Table 2 Plastic properties of sand

Material parameter	Values
Peak angle of friction	50 deg.
Peak effective plastic strain	0.05
Residual angle of friction	34 deg.
Relative Density	0.85
Shear band width	Not used
Peak dilatancy angle	15 deg.
Stress-dilatancy relation	Rows

Modeling of the pipe:

Fig. 6 shows the two generated nonlinear truss elements connected successively each other in the horizontal direction. At the left end of the two elements, a fixed hinge was given to the node. The tensile load was generated by a nodal velocity (or the tensile strain was generated by the nodal force) at the right end of the two elements.

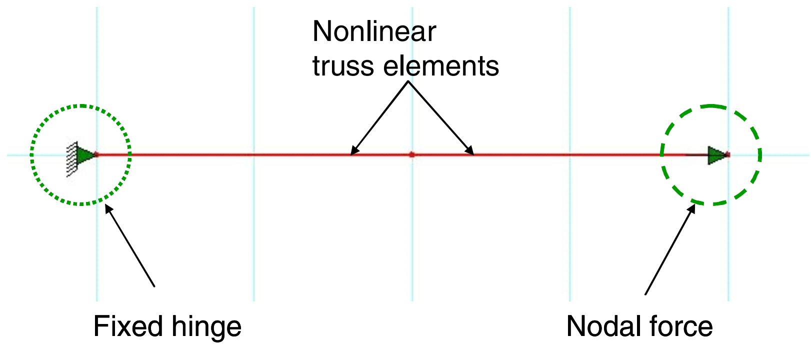

Figure 6 FEM nonlinear truss elements for the tensile loading test simulation

For the linear elastic pipe material, modulus of elasticity and Poisson's ratio was chosen as 254000 kgf/cm^2 and 0.3, respectively.

Results:

<u>One kink pipe</u>

Figure 7 plots the distribution of axial force in the pipe along the borehole obtained from the finite element analysis. The axial force increases non-linearly from left to the right. The non-linear axial force is obtained due to non-linear material model used in the analysis. Non-linearity may also arise from plastic strain of the interface and surrounding soil resulting from shear deformation. Figure 8 shows the contour of shear strain in the soil during pulling operation. Significant shear deformation is evident in soil along the length of the pipe. The strain is concentrated in the pipe vicinity. However, the strain is mainly concentrated to the pipe segment in the direction of pull, while the shear strain is negligible on the other side. The maximum pulling for the pipe was obtained as 8.97 kg-f.

<u>Two kink pipe</u>

Figure 9 shows the distribution of pipe axial force for two kink pipe along the borehole. This mechanism of pipe is commonly used HDD application. Figure 10 shows the contour of shear strain in the soil for the two kink pipe. Shear strain is distributed along the length of the pipe for this case. However, the concentration of the strain is still on the segment toward the pipe as in the case of one kink pipe. The maximum pulling for the two kink pipe was less than that for on kink pipe (i.e. 8.97 kg-f). Shear strain was distributed in a wider zone of soil for the one kink pipe. Thus, larger soil strain was needed to be mobilized, which may the reason of the higher pulling force required.

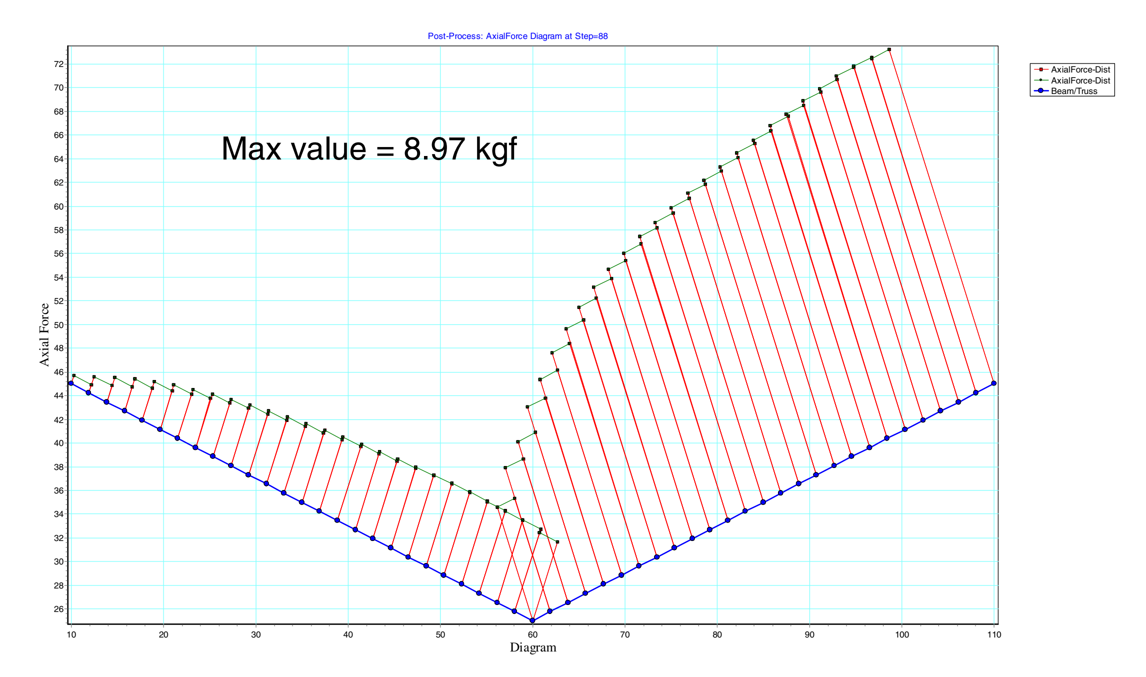

Figure 7 Axial force distribution along the pipe for one kink pipe

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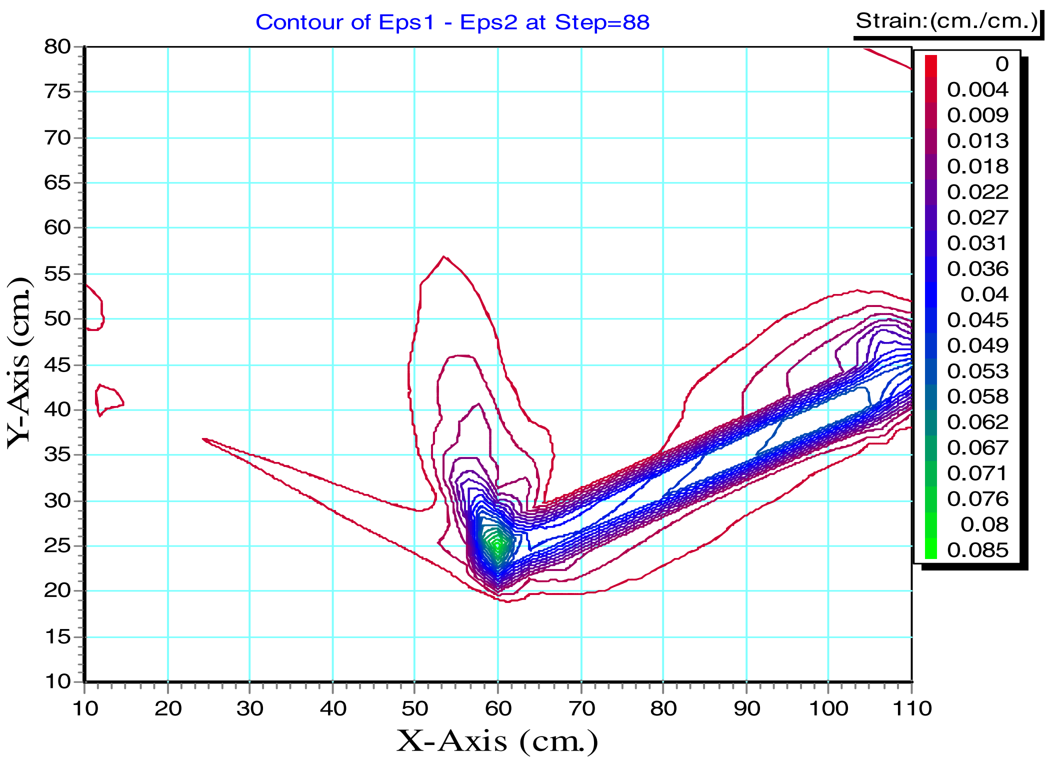

Figure 8 Distribution of maximum shear strain around the pipe for one kink pipe

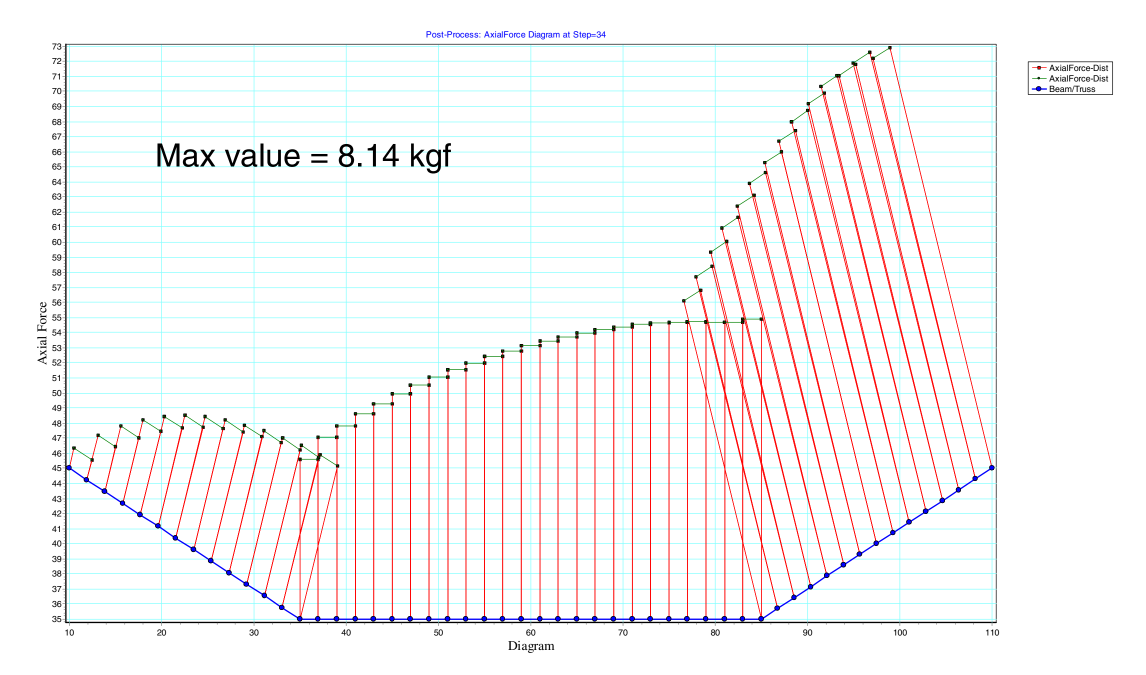

Figure 9 Axial force distribution along the pipe for two kink pipe

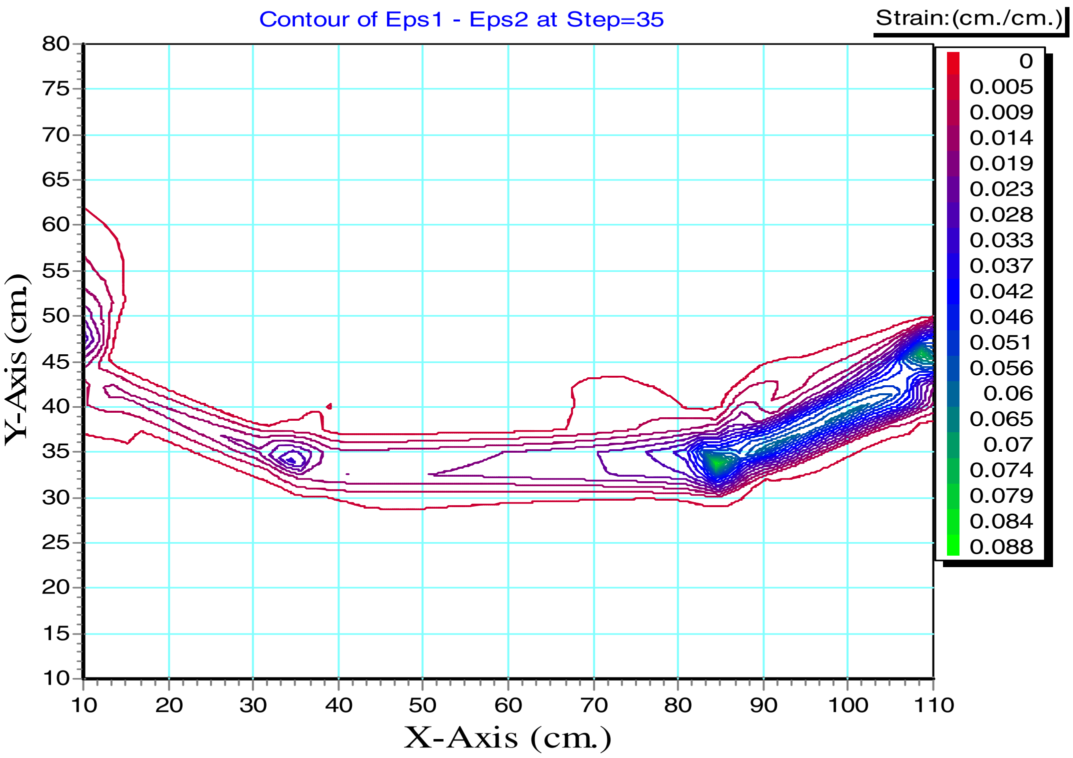

Figure 10 Distribution of maximum shear strain around the pipe for two kink pipe

Three kink pipe

Figure 11 shows the distribution of pipe axial force for three kink pipe. Maximum axial force was the lowest for this pipe among the pipe considered in this study, indicating that increase in number of kink reduce the axial force required to pull back a pipe. The maximum axial force for the pipe was obtained as 6.56 kg-f. Shear strain is distributed along the pipe length with concentration on last two segments for the pipe. Figure 12 shows the contour of shear strain in the soil for the three kink pipe. Plastic zone of soil is located in a narrower zone of soil (Figure 12), explaining the reason for low pulling required for the pipe.

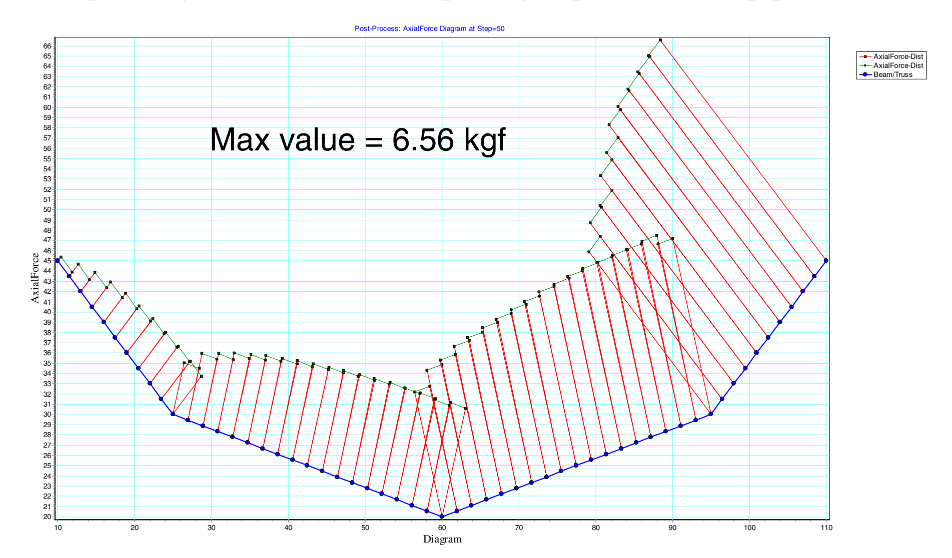

Figure 11 Axial force distribution along the pipe for three kink pipe

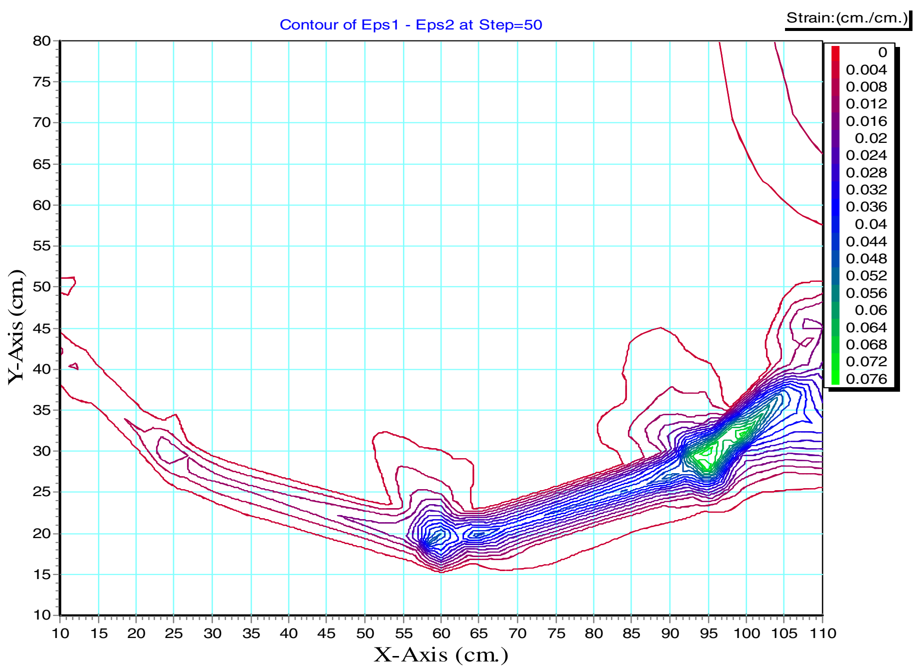

Figure 12 Distribution of maximum shear strain around the pipe for three kink pipe

It is revealed from this study that increase in the number of kink during pipe installation reduces the pulling force required to pull back a pipe due reduction of plastic zone with the increase of kink. The maximum pulling force as a function of pulling displacement is plotted in Figure 13. Figure 13 shows that displacement of the pipe is less if number of kink is increased. Maximum displacement for one kink, two kink and three kink pipe was 0.18 cm, 0.16 cm and 0.1 cm, respectively, while the maximum pulling force for the three pipes were 8.97 kg-f, 8.14 kg-f, and 6.56 kg-f, respectively. Both pipe deformation and pulling force was slightly less for the two kink pipe compared to the three kink pipe. However, both the pipe deformation and pulling force was significantly reduced if the number of kink is increased by one. This indicates that an optimum number of kink may be obtained for most efficient installation pipe using HDD. Research is underway to determine optimum borehole path and study the effects of different installation and ground parameters for pipe installation using horizontal directional drilling.

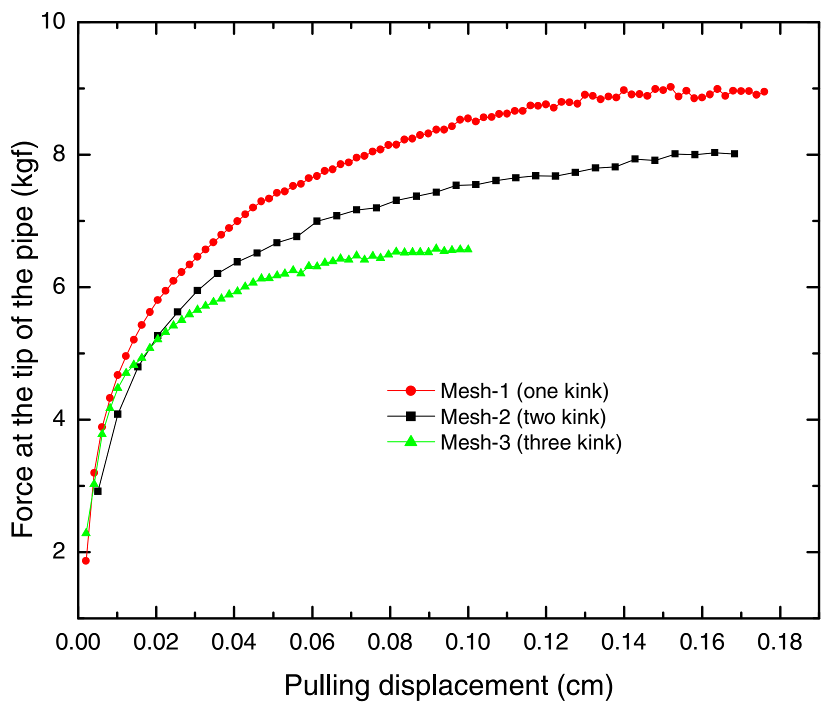

Figure 13 Variation of pulling force with pulling displacement for different meshes.

Conclusion:

Finite element analysis is used in this research to investigate few mechanisms of pipe installation during horizontal directional drilling. Pipes along three different bore-paths with one kink, two kink and three kink pipe layout was investigated. It was revealed that maximum pulling force required to pullback a pipe though the bore-path reduces with the increase of the number of kink. However, optimum number of kink may be determined for most efficient borepath. With the increase of the number of kink, the plastic zone of soil is located within a narrower zone close to the pipe, which is the reason for lower pulling force required. It is also observed that with the increase in the number of kinks in the pipe, distribution of axial force becomes less abrupt and smooth. The development of plastic zone around the pipe is more uniform in case of pipe with more kinks.

It is obvious from this study that pulling force of a pipe depend not only on the length, diameter and depth of the pipe as considered in tradition design of pipe for horizontal directional drilling. It also depends on the layout path, it follows. The pulling force was found to depend depends with the number of kinks, with probably the minimum force for a circular path.

References:

Baumert, M.E. and Allouche, E.N. (2002 "Methods for Estimating Pipe Pullback Loads for Horizontal Directional Drilling (HDD) Crossing", Journal of Infrastructure System, ASCE, Vol. 8, No. 1, pp: 12-19.

Dhar, A.S., Azad, A.K. and Islam, M.K. (2006) "Water Supply and Sewage Pipeline System in Bangladesh: Prospects of Trenchless Installation Methods", 24th International No-Dig 2006 Conference and Exhibition, Oct. 29 – Nov. 02, 2006, Queensland, Australia.

Driscopipe (1993) "Technical Expertise Application for driscopipe in directional-drilling and river-crossing", Tech. Note No. 41.

Drillpath (1996) "Drillpath theory and user's manual", Infrasoft L.L.C., Houston.
Hoque,E. and Tatsuoka,F. (1998) Anisotropy in the elastic deformation of materials", Soils and Foundations **Soils and Foundations**, Vol.38, No.1, pp.163-179.

Huey, D.P. Hair, J.D. and McLeod, K.B. (1996) "Installation Loading and Stress Analysis Involved with Pipelines Installed in Horizontal Directional Drilling", No-Dig, New Orleans.

Kotake,N., Tatsuoka,F., Tanaka,T., Siddiquee,M.S.A. and Yamauchi,H. 1999. An insight into the failure of reinforced sand in plane strain compression by FEM simulation, **Soils and Foundations**, Vol.39, No.5, pp.103-130.

Lashen, A. and Polak, M.A. (2001) "HDD loads imposed on a pipe", No Dig 2001 Conference, Nashville, Tenn.

Ortiz, M. And Simo, J. C. (1986) An analysis of a new class of integration algorithms for elastoplastic constitutive relations, **International Journal for Numerical Methods in Engineering**, Vol. 23, pp.353-366.

Peng,F.L., Kotake,N., Tatsuoka,F., Hirakawa,D. and Tanaka,T. 2000. Plane strain compression behaviour of geogrid-reinforced sand and its numerical analysis, **Soils and Foundations**, Vol.40, No.3, pp.55-74.

Pietruszczak,S.T. and Mroz,Z. (1981), "Finite element analysis of deformation of strain softening materials", *Int. J. Numer. Methods Eng*., Vol.17, pp.327-334.

Reese S, (2003), On a consistent hourglass stabilization technique to treat large inelastic deformations and thermo-mechanical coupling in plane strain problems. International Journal for Numerical Methods in Engineering ; 57: 1095–1127

Roark, R.J. (1965) "Formulas for stress and strain", 5th Ed. McGrawHIll, New York.

Rowe, P. W., (1962) The Stress dilatancy relation for static equilibrium of an assembly of particles in contact, **Proceedings of Royal Society,** London, series A, pp. 500-527.

Siddiquee,M.S.A., Tanaka,T., Tatsuoka,F., Tani,K. and Morimoto,T. (1999), FEM simulation of scale effect in bearing capacity of strip footing on sand, **Soils and Foundations***,* Vol.39, No.4, pp.91-109.

Siddiquee, M. S. A., Tatsuoka, F., Tanaka, T. , Tani, K., Yoshida, K. and Morimoto, T. (2001a); Model tests and FEM simulation of some factors affecting the bearing capacity of footing on sand, **Soils and Foundations***,* Vol.41, No.2, pp.53-76.

Siddiquee, M. S. A. And Tatsuoka, F. (2001b), Modeling time-dependent stress-strain behaviour of stiff geomaterials and its applications, **Proc. 10th International Conference on Computer Methods and Advances in Geomechanics (IACMAG),** Tucson, Arizona on january 7-12.

Siddiquee M. S. A., Tatsuoka F. and Tanaka T. (2006), "FEM simulation of the viscous effects on the stress-strain behaviour of sand in plane strain compression", *Soils and Foundations*, Vol. 46, No. 1, 99--108.

Siddiquee, Md. Saiful Alam, (2006), "A Fast Implicit Integration Scheme To Solve Highly Nonlinear System" *Soil Stress-Strain Behavior: Measurement, Modeling and Analysis Geotechnical Symposium in Roma, Italy, March 16 & 17.*

Tatsuoka,F., Uchimura,T., Hayano,K., Di Benedetto,H., Koseki,J. and Siddiquee,M.S.A. (2001), Time-dependent deformation characteristics of stiff geomaterials in engineering practice, the Theme Lecture, **Proc. of the Second International Conference on Pre-failure Deformation Characteristics of Geomaterials, Torino, 1999, Balkema** (Jamiolkowski et al., eds.), Vol. 2, pp.1161-1262.

Tatsuoka, F., Siddiquee, M. S. A., Park, C.-S., Sakamoto, M. and Abe, F., (1993) Modelling stress-strain relations of sand, **Soils and Foundations**, Vol. 33, No.2, pp. 60-81.

Vermeer, P. A. and de Borst, R. (1984) Non-associated plasticity for soils, concrete and rock, **Heron**, **29(3)**, 1-64.

Information Pipeline: Enhancing Pipeline Management & Analysis using GIS at the Tarrant Regional Water District

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Abstract

The Tarrant Regional Water District (TRWD) is a political subdivision of the State of Texas formed in 1924 with the purposes of water supply and flood control. One of the largest raw water suppliers in the state, owns four reservoirs and over 150 miles of large diameter water transmission pipeline. 1.6 million people in North Central Texas currently rely on TRWD to provide reliable, clean raw water to its wholesale customer agencies.

Engineers at TRWD discovered that the “real-world” conditions surrounding their pipelines had changed from that shown on their original “as-built” plans of construction. Additionally, routine inspection and analysis of the pipeline system and related aboveground features created numerous datasets necessary to accurately and efficiently plan capital improvement projects to maintain the pipelines and associated assets in a sustainable manner. TRWD was interested in leveraging their current investment in GIS technology to manage these critical assets and related features by utilizing the capabilities of the ESRI geodatabase.

Given the complex nature of the problem, TRWD is utilizing a wide range of resources from various internal departments (Engineering and IT/GIS) and external consultants as well. GIS staff at TRWD created a detailed geodatabase that included information on aboveground features such as land use, soil conditions, utility networks, right-of-ways, parcel ownership, and aerial photography. For the underground pipeline information, TRWD contracted with the Pressure Pipe Inspection Company (PPIC) to develop the water transmission pipeline and add these features to this geodatabase. PPIC developed a detailed water transmission data model to track individual pipe segments, from bell to spigot, with inspection, integrity and repair information. Pipe segments were positioned using the inline inspection data collected by PPIC and the differential GPS data collected on aboveground features by TRWD.

A variety of desk top and web-enabled applications have been built using ArcObjects within the ESRI GIS software, and through web browsers, to enhance the casual GIS user experience. These applications leverage off the wealth of data stored in the single geodatabase, allowing TRWD pipeline operators to fully capitalize on the advantages of maintaining their pipeline assets in a spatial database. The greatest advantage to this approach has been increased data availability to TRWD staff for-accurate and efficient identification of urgent pipeline repair needs. This has resulted in improved pipeline and associated asset management through more effective maintenance activities and the ability to achieve a more comprehensive view of the pipeline system in the capital improvement planning process.

Introduction

The Tarrant Regional Water District (TRWD) is a raw water supplier in north central Texas, serving communities in 10 counties. Its four primary customers are the cities of Fort Worth, Arlington, and Mansfield and the Trinity River Authority (TRA) as well as other surrounding communities. In all, TRWD has 26 municipal customers that, in turn, treat and distribute the water to approximately
1.6 million people. Demands in Tarrant County account for approximately 92 percent of the water supplied by TRWD.

TRWD's supply system consists of four supply reservoirs, two pipelines, and two pipeline terminal storage reservoirs. The two western reservoirs, Bridgeport and Eagle Mountain, hold approximately 20 percent of the total supply and release water down the Trinity River to various treatment plants. Water from Cedar Creek is pumped using a 72- and 84-inch diameter, 74-mile-long pipeline to the Tarrant County water treatment plants and/or terminal storage reservoirs. Similarly, water is pumped from Richland-Chambers in a 90- and 108-inch diameter, 78-mile-long pipeline to the same Tarrant County customers and storage reservoirs. Lakes Arlington and Benbrook serve as terminal storage reservoirs for the delivery system. Water from Lake Benbrook can be pumped in the reverse direction to serve the four primary customer water treatment plants when required. Water from Lake Arlington is fed via a gravity pipeline to one of the Arlington treatment plants.

The Cedar Creek and Richland-Chambers reservoirs account for 80 percent of the water supply for Tarrant County, making the respective pipeline systems' reliability essential to meeting demands. Two areas in which TRWD is utilizing GIS technologies to manage reliability and risk are pipeline soil resistivity and pipe segment integrity.

Pipe Segment Integrity

The Cedar Creek pipeline was put into service in 1972 and has experienced 10 catastrophic failures because of corrosion and embrittlement of the pipe segment prestressed wire. The Richland-Chambers pipeline was put into service in 1988 and has experienced seven catastrophic failures for the same reasons.
In 1989, TRWD began efforts to analyze and arrest the corrosion and embrittlement failure problems. A passive cathodic protection system (sacrificial zinc anodes) was installed on both pipelines to arrest deterioration of the prestressed wire. Valve controls were installed to eliminate mortar cracking caused by system surges that exposed the prestressed wire, initiating corrosion potential.

The Cedar Creek pipeline responded positively to the cathodic protection system. There have been no corrosion-induced failures since 1993, and only one hydrogen embrittlement failure (2004). The Richland line has also responded to these risk mitigation measures. The last failure on this line occurred in 1999.

In conjunction with the installation of the cathodic protection system, TRWD also undertook a series of condition assessment measures. Beginning in the late 1980's TRWD used visual inspections, sounding, acoustic monitoring and the impact-echo technique in an attempt to locate individual distressed sections of pipe. In 1998, TRWD began inspecting the pipelines utilizing Pressure Pipe Inspection Company's (PPIC) Remote Field Eddy Current/Transformer Coupling system (RFEC/TC). RFEC/TC inspections identify the number of broken prestressed wires in a given pipe segment. The system is based on electromagnetic field principles. TRWD found that this technique was effective in both identifying PCCP pipes with broken wires and quantifying the damage. TRWD has continued its use of the RFEC/TC technique, assaying a portion of the pipeline each winter. PPIC has inspected a total of 37,148 segments in the two pipelines and found 818 segments with broken prestressed wires — 718 on the Cedar Creek line and 100 on the Richland-Chambers line. The 818 segments were classified and prioritized based on the number and concentration of broken wires. Consecutive broken wires in a localized area are an indication of corrosion-related wire breaks, while intermittent breaks randomly located around the circumference may indicate embrittlement or physical damage-related wire breaks. To date, approximately 70 segments have been replaced or repaired.

Given the complex nature of pipeline deterioration, TRWD is utilizing a wide range of resources from various internal departments (Engineering, IT/GIS) and external consultants as well. To help manage the data, GIS staff at TRWD created a detailed geodatabase that included information on above ground features such as land use, soil conditions, utility networks, right-of-ways, parcel ownership, and aerial photography.

The challenge TRWD faced with managing the pipe segment inspection, classification, location, priority, and replacement information. At the conclusion of the first round of RFEC/TC inspections, PPIC had pipe segment geometry for the two pipelines in a geodatabase format. TRWD contracted with PPIC to develop a water transmission data model and add these pipe features to the geodatabase. PPIC suggested a detailed water transmission data model to track individual pipe segments, from bell to spigot, with inspection, integrity and rehabilitation information.

A Water Pipeline Data Model

To date, water network databases have not modelled pipelines to a scale incorporating individual pipe segments into the database. Generally, the pipeline network consisted of nodes and edges where a node could represent a manhole or valve and the edge is the pipeline between the manhole and valve. Information about the edge pertains to the entire length of pipeline between the two nodes. This type of database structure lends itself to a number of network modelling options and a limited amount of asset management. For example, one can perform water traces, isolate valves, or determine disconnected areas of the network. Unfortunately, detailed information about individual pipe joints is not available using this type of data model. The RFEC/TC inspection technology, for example, offers utilities information not only per pipe segment but also at regions along the pipe. Further, the electromagnetic field is sensitive to the large amount of steel present at the pipe joints and therefore clearly identifies each and every pipe section. A well-structured database containing this information would present a wealth of information to pipeline operators.

PPIC proposed a data structure where a top level, or network level, uses a standard water distribution database whereby nodes and edges represent the broader pipeline without the detailed of individual pipe segments. This level may hold information such as the type of pipeline, flow type, flow direction, etc. The level has connections with network topology, coordinates, and other geodata necessary for positional accuracy. Branching out from the network assets is information pertaining to distinct pipe segments. In this section the nodes are the ends of the pipe segment while the edge is the pipe invert. A pipe table lists each pipe segment. A table of this nature holds the elements pertaining to each pipe joint such as wire pitch, wire size, mortar thickness, bell ring thickness, etc. This table links to various types of inspection data, a coordinate system, topology, and where these pipe segments fit along the pipeline.

In contrast to traditional water network modeling, in the TRWD model, pipe sections can be tracked assets, analyzed, and modelled. Pipe failure modelling, risk management, and cost analysis in contrast to network analysis involve more detailed information provided with the new database model.

Implementation Process

A the first step in implementing this new pipe specific data model TRWD and PPIC worked together to conduct a data audit – collecting all of TRWD's diverse range of pipeline related data sets. TRWD handed over its array of lay sheets, spreadsheets and notes, which PPIC organized into a logical data structure.

As the next step, accurately positioning the pipeline required precise pipe positional data and pipe segment lay information. TRWD already had GPS co-ordinates for a variety of surface features including Air Valves, Blow Offs and Manholes, which simplified the process. Pipe segment lay information, taken from the RFEC/TC inspection results, further uniquely identified each pipe segment between these surface features. Consequently, PPIC was able to incorporate this information into a spatial database.

TRWD and PPIC worked together to record infrastructure characteristics, defects, repairs and maintenance - giving TRWD's operators a visual representation and record of the pipeline's history. Information sets incorporated include: pipe defects, length, joints, position, class, soil resistivity or chloride concentration, surface infrastructure, topography, land use, customer locations, operating pressure as well as results from RFEC/TC inspections. Completion of the pipe inventory was the most time consuming phase of the data model implementation. The database currently includes tables and fields that contain a comprehensive view of TRWD's risk management data.

Risk Management Using GIS

Risk management is the process of making and implementing decisions that will minimize the adverse effects of accidental and business losses on an organization. The decision process involves the following steps:

- Identifying and analyzing loss exposures
- Examining alternative techniques for dealing with those exposures
- Selecting the most promising techniques
- Implementing the chosen techniques
- Monitoring the results to see if the loss exposure has been dealt with most cost-effectively

Conducting a risk management assessment for a pipeline on a pipe segment by pipe segment basis requires the capabilities of a fully functioning GIS environment because of the large amounts of data and computational time required. Determining the change in pre-stressing wire breaks over time, for example, is a time-consuming task that is easily queried and displayed within a GIS.

Changes in pre-stressing wire breaks between inspections is a relatively simple procedure in a GIS that would take a long time to prepare without the use of a GIS given the number of pipe segments per mile of pipeline. Calculating which pipes will fail is a more complicated study because of the many different variables involved other than the number of pipe breaks. Manually, computing multiple variables over a few miles of pipeline is an imposing task. The GIS functions simplify even the most complicated failure scenario by using algorithms to calculate those pipes with the highest degree of failure increase the accuracy of the model and offer quick analysis and multiple scenarios.

Case Study – Soil Resistivity Analysis Using GIS

Soil resistivity readings measure influential properties about the soil. With respect to pipelines, soil resistivity is an indicator of potential corrosion. Identifying and monitoring soil resistivity along the right-of-way (ROW) allows TRWD to design, install, operate, and maintain an optimum cathodic protection system. The protection can be tuned to the soil conditions. By mitigating the corrosion potential around the pipelines, system reliability is increased.

To accomplish this, soil resistivity measurements are taken at 200-foot intervals along the pipelines within the ROW. The readings are then converted and used to identify the soil resistivities at discrete 5-, 10-, 15-, and 20-foot depths, then extrapolated to provide an estimate per layer to describe soil resistivities at 0–5-foot, 5–10-foot, and 15–20-foot depth ranges.

The soil resistivity data is then used to create a route. Each of the measurement locations is located along the route. Using the linear referencing capabilities in GIS, the distances held in a database table are used to accurately place the monitoring points along the route. Once these locations are placed, a 100-foot buffer is created around each monitoring location.

A spatial join procedure is then conducted to transfer the attribute values of each monitoring location to the 100-foot buffer area. Soil resistivity values have been classified in the five-foot layer ranges for each monitoring location. The corresponding 100-foot buffer zone around each monitoring location has also been classified.

Once the attributes of the monitoring locations have been attached to the buffer, another spatial join procedure is conducted to join all of the soil resistivity measurements contained within the 100-foot buffer zone to each pipe segment. The color of each pipe segment with respect to the transparent 100-foot buffer zone indicates that the pipe segments, buffer zone, and monitoring location have the same soil resistivity values. Once classified, the datasets are overlaid on top of the aerial images.

Custom Applications

Not every staff member at TRWD is an expert GIS user and not every staff member will use GIS on a daily basis. To enhance the casual GIS user experience, a variety of desktop and web-enabled applications have been built within the GIS environment. These applications leverage off the wealth of data stored in the single geodatabase, allowing TRWD pipeline operators to fully capitalize on the advantages of maintaining their pipeline assets in a spatial database. The applications include the ability to allow the user to:

- Identify urgent repair needs;
- Query the database to locate pipes that possess a specific attribute, or combination of attributes;
- View these attributes, as well as any other data set, documentation, image, or historical record, for an individual pipe;
- Produce graphs and charts that are based on any combination of fields or tables contained within the database.

These applications and others allow the GIS system to accurately show pipeline and pipe segment details: segment length, depth, orientation, bends, specials, and appurtenances. These same details will also be updated as pipe segment replacements are completed along with additional first-order GPS coordinate information for future reference. In addition, the GIS will be used to identify which pipe segments should be replaced to reduce the risk of failure. Attributes such as soil resistivity, residual remaining strength, rate of deterioration, operating pressure, transient pressure, adjacent land use, and pipe polarization will be combined into a risk model that will determine pipe segment replacement priority.

Summary

The current value of TRWD's pipeline infrastructure is approximately $300 million. The average pipe segment repair cost is $40,000. Replacing the 300 highest-priority segments would cost at least $12 million. The primary objective of the GIS project is to provide current pipeline systems assessment and life cycle cost analysis information and to manage risk by prioritizing pipe replacement over the next 20 years to reduce the impact on operations budgets and downtime. The greatest advantage to the GIS approach has been increased data availability to TRWD staff for accurate and efficient identification of urgent pipeline repair needs. This has resulted in improved pipeline and associated asset management through more effective maintenance activities and the ability to achieve a more comprehensive view of the pipeline system in the capital improvement planning process.

Failure Risk Analysis of Lined Cylinder Pipes with Broken Wires and Corroded Cylinder

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Abstract

Nonlinear finite-element analysis is used to evaluate the performance of lined-cylinder prestressed concrete pressure pipe with broken wires and thinning of the steel cylinder due to corrosion. The model includes nonlinear stress-strain relationships to represent yielding in steel and compressive crushing, tensile softening and cracking in concrete. A relationship between prestress loss length and the shape and the variation of thinning in steel cylinder was developed from inspection of failed and nearly failed pipes in a line. This relationship is used in the finite element model to represent the effects of corrosion of the steel cylinder in the performance of the pipe. Analyses are conducted for pipes with different prestress loss lengths. In each analysis, the pipe is initially subjected to prestress, self weight, water weight, and earth load. After the application of gravity loads, the internal pressure of the pipe is gradually increased to a point at which the radius of the pipe increases without any significant increase in the internal pressure. Analysis results show crack patterns, crack width, crack depth, pipe deflection, change in the stress in the wires, and concrete crushing as a function of prestress loss length. These results are used to define engineering criteria for risk analysis and repair priorities determination of pipes with broken wires.

Introduction

There are two types of PCCP in use, 1) Embedded Cylinder Pipe (ECP), and 2) Lined Cylinder Pipe (LCP). This paper focuses on the structural performance of distressed LCP.

Degradation of LCP consists of corrosion of prestressing wires and the steel cylinder, cracking of the concrete core, and cracking and debonding of the mortar coating. The excessive corrosion in prestressing wires can cause wire breaks in an area referred to as broken wire zones (BWZ). The corrosion also causes thinning and in the most severe cases perforation of the steel cylinder. The region where thickness of steel cylinder is reduced due to corrosion is called the thinning zone. As these degradations take place, the pipe starts to leak and in the ultimate case ruptures causing significant time and labor loss to the owner.

As with all engineered systems, continuous monitoring and rehabilitation of pipelines is essential to prevent such undesirable and costly failures. With the advancement in nondestructive condition assessment technologies, such as detection of number of broken wires (NBW) with the Remote Field Eddy Current/Transformer Coupling (RFEC/TC) method (Mergelas and Atherton, 1998), the pipeline owners are provided cost effective and accurate health monitoring methods. Even though these technologies provide an accurate picture of the amount of degradation (such as NBWs in a given pipe) in the pipeline, they do not provide any information on risk of failure in the pipeline as a function of the measured quantities. Such information is essential to optimize rehabilitation strategies as well as to lay-out the sequence of the rehabilitation work.

The failure risk of both ECP has been extensively studied by Zarghamee et al (2003). The structural performance of distressed LCP differs from ECP in a major way. In LCP, the steel cylinder is outside of the core and corrosion of wires is accompanied by pitting, corrosion, and thinning of the steel cylinder. In contrast, corrosion and breakage of wires in ECP does not initially accompany corrosion and weakening of the steel cylinder until after several years beyond the stage where wide cracks develop in the outer core providing access for the corrosive environment to the steel cylinder embedded in the core.

This paper discusses the analytical studies conducted to develop a tool (risk curves) that can be utilized to evaluate the risk of failure in lined cylinder pipe (LCP) with broken wires and corroded steel cylinder.

Method of Approach

Structural performance of LCP for a given level of distress, usually detected by the RFEC/TC method, is evaluated by comparing the maximum pressure in the pipe to the pressures that cause various levels of damage (limit states) in the pipe. For this purpose, a set of risk curves was developed showing the relationship between various limit states and the maximum pressure inside the pipe for a given number of broken wires in the pipe. The damage states that are considered in this study are as follows:

Serviceability Limit State:
- Onset of first longitudinal cracking in the concrete core.

Damage Limit States: Damage limit state is the minimum of the following criteria
- Increase in wire stress of 30 ksi adjacent to BWZ.
- Cylinder yielding in the nonlinear finite element model where cylinder thickness is equal to the nominal thickness.

Strength Limit States: Two groups of criteria are selected. The first group of criteria, referred to as Strength Limit State I, is the minimum criterion calculated for a pipe wall in which internal pressure is applied to inner core and consists of the following limit states:

- The compressive strain in concrete core reaches 3,000 microstrains.
- Stress in the wires adjacent to BWZ in the nonlinear finite element model reaches 95% of the ultimate strength of the prestressing wires.
- Steel cylinder ultimate strength after hardening in the nonlinear finite element model anywhere where cylinder thickness is equal to the nominal thickness.

The second group of criteria, referred to as Strength Limit State II, is based on the assumption that the steel cylinder has expanded and water can freely flow behind the core so that internal pressure is applied entirely to the steel cylinder and the concrete core is stress free. This is a conservative assumption and consists, in addition to the above three limit states, of the following additional limit states:

- Steel cylinder ultimate strength based on the strength of pitted and thinned steel pipe alone subjected to internal pressure, i.e., assuming that the core is cracked to an extent that pressure is now applied entirely to the steel cylinder. Method of prediction of strength based on procedure developed by Loureiro et. al. (2001).
- Cylinder ultimate strength considering thinning due to corrosion and bi-axial stress state due to bulging of steel cylinder under pressure.

The above limit states divide the plots of pressure versus number of broken wires into four different zones, as shown in Figure 5. Each zone is assigned an alphanumeric order, depending on the risk of pipe failure and the need for repair. The structural condition of a given distressed pipe for each repair priority is described below. These repair priorities are used to determine the risk of failure in a given distressed pipe.

Priority 1: The maximum pressure in the line exceeds the pressure that produces the Strength Limit State II with contribution of soil resistance (Priority 1a) and Strength Limit State I with contribution of soil resistance (Priority 1b) and Strength Limit State I without soil resistance (Priority 1c). The failure can occur at any time. Repair should be performed as soon as practical.

Priority 2: The maximum pressure exceeds the minimum pressure that produces damage limit states, but not any of the strength limit states. The failure can occur with time as the number of broken wires increases, or when the steel cylinder corrodes. Repair should be performed within a short time period.

Priority 3: The maximum pressure in the line exceeds the pressure that produces serviceability limit state, but not the damage limit state. The failure of the pipe, if it occurs at all, can occur after a much longer time period than in Priority 2. The pipeline should be monitored.

Priority 4: The maximum pressure in the line is less than the pressure that produces serviceability limit states. The failure of the pipe is not expected in the near future.

Finite Element Modeling

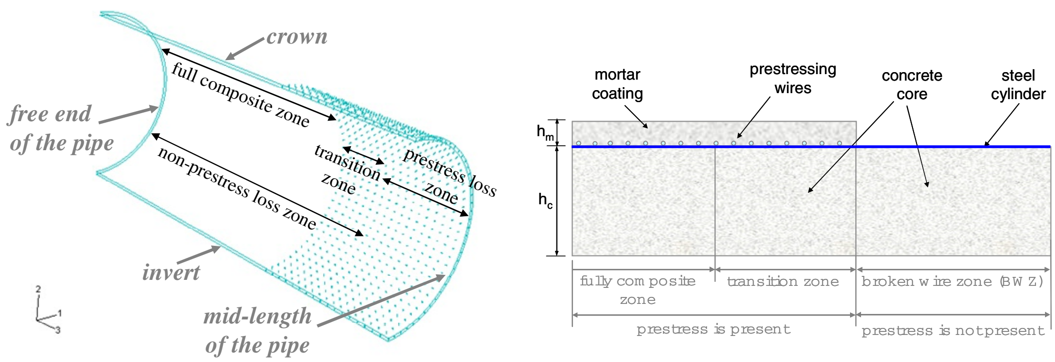
A set of nonlinear finite element (FE) analyses were conducted to determine the pressures corresponding to various limits states of LCP with different BWZ lengths. In this study, the FE analyses were performed with a 48 in. diameter LCP. Geometric properties of the pipe analyzed in this study are summarized in Table 1.

Table 1. Geometric properties of the pipe analyzed in this study.

Length (ft)	Inner diameter (in.)	Core wall thickness (in.)	Steel cylinder thickness (in.)	Mortar thickness (in.)	Wraps per foot	Prestress wire area (in.^{2}/ft)
20	48	2.94	0.0598	0.75	26.6	0.549

The FE model included the effects of pipe weight (680 lb/ft), fluid weight (784 lb/ft), and earth load (6,310 lb/ft for 8 ft of soil cover) on the pipe. After the application of these loads, the internal pressure is increased until previously defined limit states were reached. The analyses were conducted for pipes with NBW equal to 0, 15, 25, 45, 70, and 106.

Nonlinear finite element analyses were performed using ABAQUS 6.5-6. The finite element model consists of approximately 2.0 in. by 2.3 in. shell elements and represents a quarter of the pipe. The model has symmetry boundary conditions at mid-length, the crown and the invert of the pipe.



a) Different zones considered in FEA modeling of distressed PCCP

b) Different zones in PCCP shown in pipe wall cross-section

Figure 1 – Finite Element Modeling of Distressed PCCP

Figure 1 shows the conceptual model of distressed LCP used in the finite element modeling. The pipe is divided into three zones representing different states of pipe along its length. These zones are defined as: 1) the full composite zone, 2) the broken wire zone (BWZ), and 3) the transition zone.

The full composite zone represents the undamaged length of the pipe where full composite action of the cross section is expected.

The BWZ represents the length of the pipe where prestressing wires are broken. This zone has only the concrete core and the steel cylinder as the structural components; the mortar layer is conservatively assumed to be cracked and not contributing to the strength of the pipe. Furthermore, due to the absence of prestress, the steel cylinder is assumed to be debonded from the concrete core. The BWZ also contains a sub-zone termed as the “thinning zone”. This zone is created to represent thinning of the steel cylinder due to corrosion. The approach we used in defining the thinning zone is described in the next section.

The transition zone represents the transition from the broken wire zone to the full composite zone. In this zone, the steel cylinder and prestressing wire coil can slide relative to the concrete core. This sliding between the steel cylinder and the core takes place over a finite development length of the pipe beyond which the pipe starts to act as fully composite. For the 48 in. diameter LCP, the length of the transition zone was determined by calculating the sliding friction force equal to the yield strength of steel cylinder of 14 in.

The material properties used in the finite element analyses are summarized in Table 2 and Table 3. A nonlinear stress-strain relationship was used for each material. Concrete was modeled using smeared cracking formulation.

Table 2 – Material properties of concrete core and mortar coating.

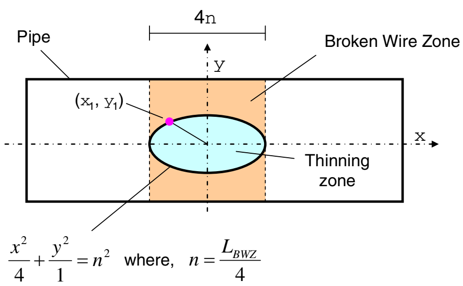
Property	Core	Mortar Coating
Compressive strength (psi)	6,000	5,500
Concrete density (lb/ft^3)	145	140
Modulus of elasticity (psi)	3,943,000	3,643,000
Poisson’s ratio	0.17	0.17
Maximum tensile stress (psi)	542	519
Micro cracking strain (μs)	137	142
Visible cracking strain (μs)	1510	1140

Table 3 – Material properties of steel cylinder and prestressing wires.

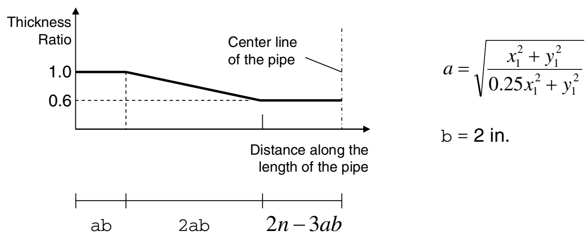
Property	Cylinder	Prestressing Wire
Modulus of elasticity (psi)	30,000,000	28,000,000
Poisson’s ratio	0.3	0.0
Density (lb/ ft^3)	489	489
Yield stress (psi)	33,000	196,350
Thickness/Diameter (in)	0.0598	0.162
Minimum tensile strength (psi)	52,000	231,000

Modeling of Thinning in Steel Cylinder

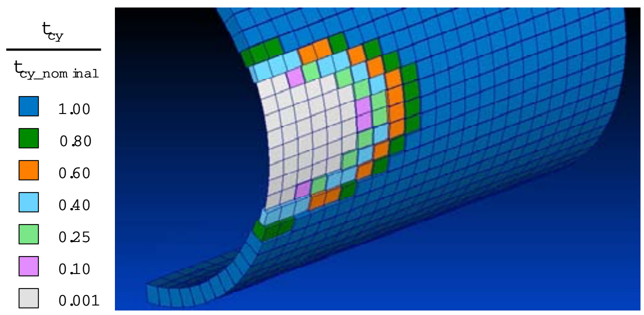
The factors causing the prestressing wires to corrode and break also cause the steel cylinder to corrode in the BWZ. Corrosion in the steel cylinder causes thinning, and in the extreme cases perforation of steel cylinder wall. The extent of thinning of the steel cylinder at rupture varies from pipeline to pipeline and depends on both internal pressure and environmental corrosivity. To define the shape and the amount of thinning in the steel cylinder, several LCPs that had experienced rupture or leakage and had been removed from the line were inspected. In each case the thickness of steel was measured after removing the corrosion products on the steel cylinder using ultrasonic measurement to measure the residual thickness of the corroded and pitted steel cylinder. These inspections and measurements suggested that at rupture, the steel cylinder was corroded and pitted, sometimes perforated, with an average thickness of about 0.035 in. (about 60 percent of the nominal thickness of 0.0598 in.) over a length that is typically smaller than BWZ.



a) Boundary of the thinning zone



b) Shape function



c) Thickness variation for L_{BWZ} = 48 in.
(note that thickness of steel cylinder is artificially increased for clarity)

Figure 2 – Modeling Thinning Zone in Steel Cylinder

Based on field inspections, the shape of the thinning zone was selected as an ellipse with a 2:1 diameter ratio and the longer diameter equal to the width of the BWZ. The variation of steel cylinder thickness inside the ellipse was defined as shown in Figure 2a and Figure 2b. Figure 2c shows the variation of thickness within the thinning zone for a pipe with 4 ft of BWZ length (70 broken wires). This thinning zone was incorporated into the FE model to represent thinning in the BWZ.

Results of Finite Element Analyses

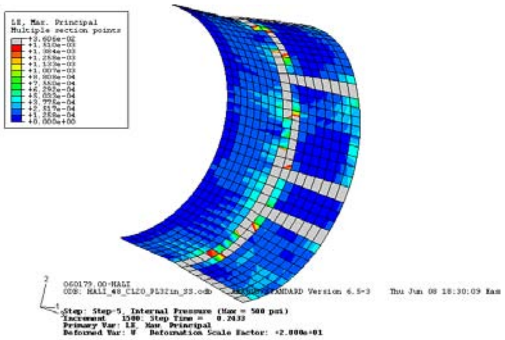
The finite element analyses were used to develop the risk curves for various limit states in the pipe. The results were also used to understand the sequence and location of cracks formed as a function of internal pressure.

Crack formation sequence: The length of the broken wire zone and the intensity of corrosion in the steel cylinder affected the sequence of crack formation, and the size, frequency and stability of cracks. A representative crack pattern for pipe with 70 number of broken wires at an internal pressure of 125 psi is shown in Figure 3. In this figure, the maximum value of the contours was set equal to the strain value corresponding to the onset of visible cracking (1510 microstrain). Any strain value greater than the visible cracking strain is shown with a grey color. This color coding depicts the major cracks formed at the ultimate stage of the pipe.

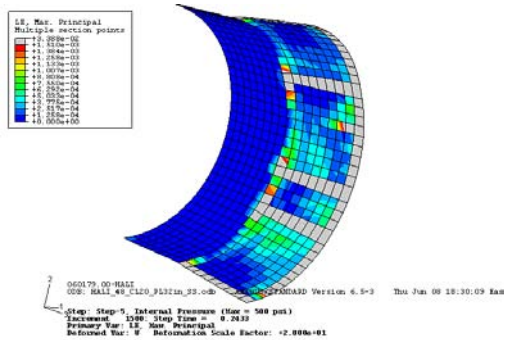
Pipes with BWZ of up to about 1 ft long (NBW = 15 and NBW = 25), showed invert cracks first, followed by longitudinal cracks at the crown and near the springline, and eventually additional longitudinal cracks between already developed cracks. In pipes with relatively small BWZ (NBW less than about 25), circumferential cracks did not develop. The longitudinal cracks in the concrete core have a maximum crack width of about 20 mils.

Pipes with BWZ between about 1.5 ft and 2.5 ft (NBW equal to 45, and 70), where the steel cylinder corroded and thinned, showed the concrete core cracked first at invert followed by two longitudinal cracks around 2 and 4 o'clock positions. Then a longitudinal crack formed at the crown followed by a longitudinal crack at the springline. At the same time, two circumferential cracks started to develop. The first circumferential crack developed in the inner face of concrete core at the section where the BWZ ends. The second circumferential crack developed in the outer face of concrete core at the mid-length of both pipes. These circumferential cracks typically grew from invert and springline and eventually joined. The longitudinal cracks have a maximum crack width of about 80 mils.

Pipe with BWZ length of about 4 ft (NBW equal to 106), which contained a large thinning zone, showed initially a longitudinal crack at invert, followed by two longitudinal cracks around 2 and 4 o'clock positions, then longitudinal cracks at the springline, with circumferential cracks at the mid-length and at the end of the BWZ. After the formation of circumferential cracks, a longitudinal crack formed at the inner surface of the concrete core at the crown. The longitudinal cracks had a maximum crack width of about 90 mils.



a) Maximum Principal Strain at Inner Surface of the Concrete Core

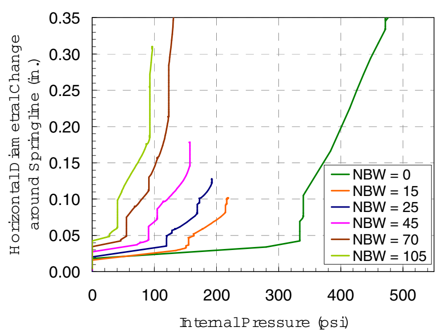


b) Maximum Principal Strain at Outer Surface of the Concrete Core

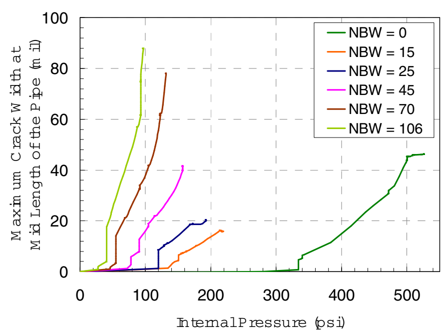
Figure 3 - Cracks in concrete core for pipe with NBW = 70

Pipe deformations and maximum crack width: Figure 4a shows the maximum horizontal change of diameter around springline with increasing internal pressure, and Figure 4b shows the maximum crack width at mid-length of the pipe with increasing internal pressure.

In the linear elastic range, the diameter of the pipes proportionally increases with the increase in the internal pressure. With the formation of cracks, the diameter increases rapidly (see sudden jumps in the curves) and continues to increase at a much faster rate than in the linear elastic range.



a) Maximum Diametral Change around Springline



b) Maximum Crack Width at Mid-Length of the Pipe

Figure 4 – Change in diameter and width of cracks as a function of internal pressure.

The maximum crack width plot (Figure 4b) has a similar slope as the displacement plot for the pipes with NBW equal to and more than 45. For these pipes, after a specific pressure, the crack width increases without significant increase in the pressure. This indicates that the pipe deformation is confined into few cracks. The maximum crack size for the pipes with NBW less than 45 does not change rapidly with increasing pressures even though the diameter of these pipes change rapidly.

This is due to formation of multiple longitudinal cracks around the circumference of the pipes in the BWZ. The formation of multiple cracks limits the size of the cracks and hence increases the pressure at which these pipes start to leak.

Risk Curves: Using the criteria defined earlier for different limit states, risk curves representing the relationship between the maximum pressure, number of broken wires were developed (Figure 5). These curves can be used to estimate the repair priority of a pipe. This can be performed by entering the graph from both axes, i.e. for a given pressure level and for a given level of distress (NBW). The location of the point on the graph determines the level of failure risk in that pipe. For example, for a 48 in. diameter pipe with 150 psi internal pressure and 30 broken wires, it can be shown that the damage state of this pipe is less than the condition defined for "Damage Limit State". The pipe is in Repair Priority 3 with low risk of failure.

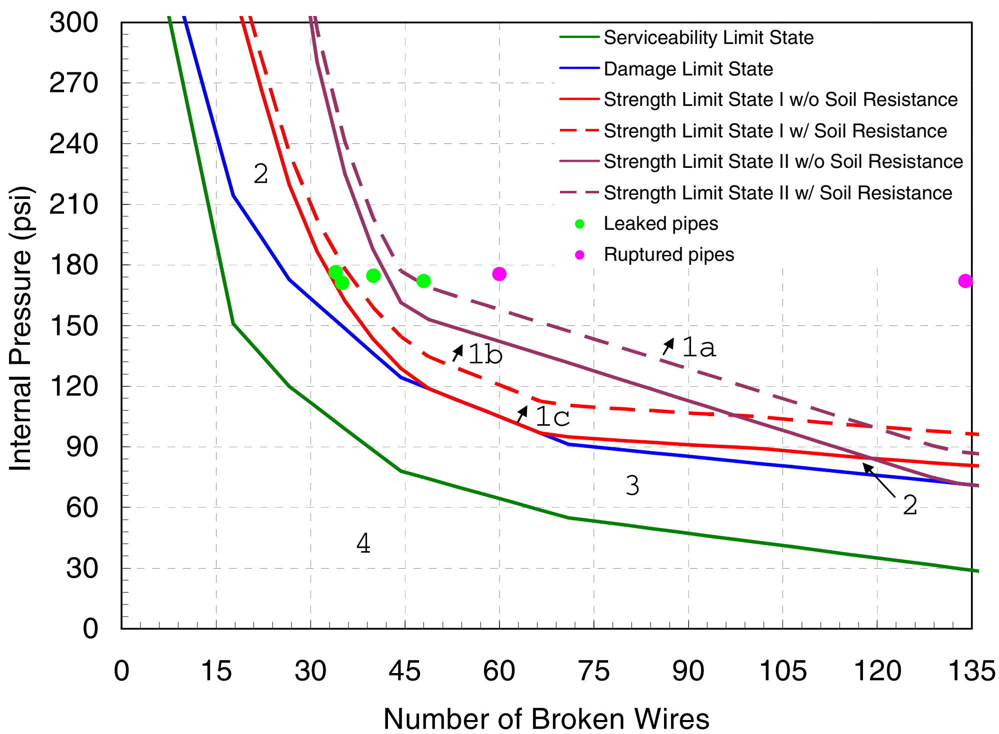


Figure 5. Risk curves developed for 48 in. lined cylinder PCCP with broken wires and corroded steel cylinder.

In Figure 5, the data (green and pink colored dots) obtained from actual pipe condition assessment surveys are also plotted. The data included four leaking pipes and two ruptured pipes. The design of these pipes were identical to the pipe analyzed in this study. The comparison of the pipe data with the generated risk curves indicate that the risk curves provide a good estimate of the pressures corresponding to different limit states for the leaked pipes. The risk curves typically underestimate the pressures corresponding to rupture. This might be attributable to the uncertainty in estimating the number of broken wires from the photos which were taken after the full rupture took place. In these pipes, the number of broken wires may have been less at the instant of rupture and may have been increased as the rupture progressed. Less number of broken wires would move the data points toward to the risk curves and hence would result in better correlation.

Conclusions

This paper presents a procedure and a graphical tool for determination of failure risk of distressed LCP with distress manifested as broken wires and thinning in the steel cylinder. The procedure and the graphical tool can be used together with emerging non-destructive technologies for detecting degradation level in LCP to identify pipes that are in high risk of failure in a given pipeline. Identification of pipes with high risk of failure can help development of repair strategies and scheduling of the rehabilitation work. With using the procedure presented in this paper, a set of similar risk curves can be constructed for different diameter pipes with different characteristics and environmental conditions.

The comparison of the developed risk curves with the actual pipe data indicated that the risk curves provide a good estimate of the pressures corresponding to different limit states.

In determining risk curves, Strength Limit State II is more conservative and may be used for risk analysis of distressed LCP. However, for many pipelines, such conservatism may be too excessive and unacceptable. Our field observation supports that failure of distressed LCP is governed by Strength Limit State I. The determination of whether Strength Limit State I should be used in risk analysis should be based on hydrostatic testing of distressed pipe.

References

Mergelas, B. J., Atherton, D. L. (1998) "Nondestructive Testing of Pre-Stressed Concrete Embedded-Cylinder Pipe Using An Inline, Electromagnetic Inspection Tool," ASCE Pipeline Division Specialty Conference, Pipelines in the Constructed Environment, San Diego, CA.

Loureiro J. F., Netto, T. A., Estefen, S. F. (2001) "On the Effect of Corrosion Defects in the Burst Pressure of Pipelines," *Proceedings of OMAE'01 20th International Conference on Offshore Mechanics and Arctic Engineering*, Rio de Janeiro, Brazil.

Zarghamee, M.S., Eggers, DW, Ojdrovic, R.P., Rose B. (2003) "Risk Analysis of Prestressed Concrete Cylinder Pipe with Broken Wires," Proceedings of ASCE Specialty Conference Pipelines, ASCE Specialty Conference, *Baltimore, Maryland*

LONG-TERM PLASTIC PIPE STIFFNESS MEASURED BY CONVENTIONAL AND ACCELERATED PROCEDURES

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Abstract

Parallel-plate loading mechanism (ASTM D2412 standard test method) was used for investigating the long-term pipe stiffness values of HDPE, PVC and ABS pipes. The conventional and accelerated test procedures were both used. The nominal inside diameters of the test pipes were 300 and 400mm. S-type long-term deflection curve was observed for the test plastic pipes on a semi-log scale. Long-term pipe deflection is a function of pipe material properties, pipe geometry, and external loading conditions. Load deflection rate approximately increases as increasing the pipe diameter. However, the relationship between load-deflection and different plastic pipe materials is still unclear. Long-term pipe stiffness values decrease as increasing test duration on a semi-log scale. The elevated temperature test procedure can be used as an accelerated method for estimating the long-term pipe deflection and pipe stiffness value in conjunction with the parallel-plate loading mechanism.

Introduction

Many of today's plastics were developed during and just before World War II. Some were introduced into piping systems in the 1930's. Plastic piping systems obtained wide acceptance in the late 1950's and early 1960's. Since then plastic pipe usage has increased at an astounding rate. The primary benefits associated with all of these plastic piping products are the following: sustainability, corrosion resistance, chemical resistance, low thermal conductivity, flexibility, low friction loss, long term performance, light weight, variety of jointing methods, nontoxic, biological resistance, easy identification, low maintenance. In general, thermoplastic piping is relatively flexible as compared to metal (rigid) piping. There are several types of thermoplastics, which are commonly used in the manufacture of pipe, such as Polyvinyl chloride (PVC), Acrylonitrile-butadiene-styrene (ABS), Polyethylene (PE), Polybutylene (PB), and Polypropylene (PP). Water mains, hot and cold water distribution, drain, waste, and vent (DWV), sewer, gas distribution, irrigation, conduit, fire sprinkler and process piping are the major markets for plastic piping systems throughout the world. Underground piping makes up the largest part of the market.

Three parameters are most essential in the design or the analysis of any flexible pipe installation. They are load (depth of burial), soil stiffness in pipe zone, and pipe stiffness. Among of them, external loads and surrounding soil stiffness are controlled by site conditions. However, pipe stiffness is closely related to material properties and is an important design parameter.

Flexible Pipe Design

In the late 1920’s and 1930, Spangler developed a rational design procedure to predict the deflection of an installed flexible conduit. This design procedure calculated the horizontal deformation of the conduct as a function of the vertical load, the bedding support provided, and soil pressures acting laterally to resist the horizontal movement of the pipe. Based upon the assumption the pipe was sufficiently rigid, the deformed shape would be that of an ellipse, the vertical deflection assumed to be approximately equal to the horizontal. Therefore, the Spangler Equation is modified as follows:

$$\Delta x = \Delta y = D_e K W_c /(0.149 PS + 0.61 E') \quad (1)$$

where:

Δx = horizontal deflection of pipe (mm),
Δy = vertical deflection of pipe (mm),
D_e = deflection lag factor,
K = bedding constant, dependent upon the support the pipe received from the bottom of the trench,
W_c = vertical load per unit of pipe length (N/m),
PS = $F/\Delta y$ = pipe stiffness (kPa),
E' = modulus of soil reaction (kPa).

As shown in the equation, pipe approximate deflections under earth load are a function of pipe stiffness, soil modulus, and the handing and installation characteristics of a pipe during the very early stage of soil around the pipe.

The EI of a pipe is a function of the material's flexural modulus (E) and the wall thickness (t) of the pipe. Since $I = t^3/12$. As such it is a fixed value for any given set of material and dimensional parameters. However, the quantities pipe stiffness (PS) and stiffness factor (SF) are computed values determined from the test resistance at a particular deflection. These values are highly dependent on the degree of deflection, for as the pipe deflects the radius of curvature changes. The greater the deflection at which PS or SF are determined, the greater the magnitude of the deviation is from the true EI value. By application of the correction factor $C = [1 + (\Delta y/2d)]^3$, the measured PS or SF values can be related to the true EI of the pipe as long as the pipe remains elliptical. Therefore:

$$PS = F C / \Delta y = F/\Delta y (1 + \Delta y/2d)^3 \quad (2)$$
$$SF = EI = 0.149 r^3 (PS) \quad (3)$$

where:

D = initial inside diameter,
r = mean radius of the pipe.

Typically, pipe stiffness (PS) at 5 and 10 % deflection, is determined for plastic pipe for each specimen. As specially request, stiffness factor (SF) also can be calculated at 5 or 10 % deflection for each specimen. At present, the ASTM D2412 is the most common used test standard. However, PVC sanitary sewer lines found to continuously deflect 13 years installation (Rinker, 2004). The long-term deflection of plastic pipes installed deep in the ground is always a concern for sewerage engineers. According to Modified Spangler's equation, flexible pipe deflection is a function of pipe stiffness. Therefore, the long-term deflections of HDPE pressure pipe, PVC and ABS jacking pipes under parallel-plate loading mechanism were investigated. Moreover, the change of pipe stiffness versus time for these pipes were also determined based upon the test results.

Test Pipes and Equipments

HDPE smooth wall pipe are commonly used for water mains application. PVC and ABS jacking pipes are the most common used pipes in sewerage system in Japan and Taiwan. Due to the flexibility of these pipes, pipes with diameter of 200 to 450 mm are used in sewerage construction. Moreover, pipes with diameter of 300 and 400 mm are the most common used pipes used in jacking pipe construction. Therefore, 300 and 400 mm HDPE, PVC and ABS plastic pipes were selected in the study. The typical physical properties of these pipes are listed in the Table 1. Pipe stiffness (PS) values of these pipes were determined according to ASTM D2412 standard test method. The initial pipe stiffness (Initial PS) will be used as the reference data for further analyses. Typical material properties of these pipes are summarized in the Table 2. These data were the average values based upon 6 sets of tests.

In order to measure the long-term vertical deflection of the test pipe under constant load, a custom-made parallel-plate loading apparatus was designed and built. Dead load was used in the system. A double balance beam system was used to transfer the dead load to the parallel load plates. The transfer load ratios were 8 to 1 and 2.5 to 1, respectively. The total transfer ratio is 20 to 1. A load cell can be attached to the test apparatus to monitor the actual loading during the test as needed. Vertical deflection was measured using a LVDT with accuracy of 0.01mm. The maximum allowable deflection is 40 mm. 12 units of this apparatus were built for conducting the standard long-term tests. These apparatuses were placed in a temperature and humility controlled room. A closer view of the setup of these test apparatuses is shown in the Figure 1. A diesel electrical generator is connected to the electricity system to prevent any interruption of the test due to the electricity supply. HOBO U12 Temp/RH external data logger is used to trace the condition of the room. All the data were collected in a data acquisition system.

Table 1 Typical physical properties of the test plastic pipes

Type	Nominal Diameter (mm)	Diameter (mm)		Thickness (mm)	Mass per Length (kgf/m)
		Outside	Inside		
PVC	300	317.70	285.30	16.03	21.30
	400	422.21	378.71	21.38	38.59
ABS	300(I)	316.34	290.61	12.84	12.57
	300(II)	327.25	299.67	13.92	14.23
	400	442.29	401.69	19.88	27.55
HDPE	300	317.03	275.83	19.62	26.72
	400	403.96	358.80	23.84	17.31

Since conducting the standard long-term test is a very time consuming process to achieve the desired test results. A series of accelerated tests was also performed. The custom-made parallel-plate loading apparatus was placed in a temperature controlled chamber. The accelerated parallel-plate load tests were performed using different elevated temperatures up to 60°C. The setup of the accelerated test apparatus is showed in the Figure 2. Based upon the test results, the relationships between pipe stiffness and test duration could be expressed in the form of one of the following equations for predicting the long-term pipe stiffness or test duration based upon the accelerated test results (Koerner et al. 1990):

$$\text{Log}\,(t_f) = A_0 + A_1\,T^{-1} + A_2\,T^{-1}\,P \qquad (1)$$

$$\text{Log}\,(t_f) = A_0 + A_1\,T^{-1} + A_2\log(P) \qquad (2)$$

$$\text{Log}\,(t_f) = A_0 + A_1\,T^{-1} + A_2\,T^{-1}\log(P) \qquad (3)$$

Where:

t_f = test duration,

T = test temperature,

P = test load,

A_0, A_1, and A_2 = constants.

Figure 1 Setup of the long-term pipe stiffness test apparatuses.

Table 2 Typical material properties of the test plastic pipes

Type	Nominal Diameter (mm)	Tensile Strength (MPa)	Ovality (%)	Initial Pipe Stiffness (kPa)
PVC	300	52.31	0.27	2092.04
	400	50.88	0.89	2069.73
ABS	300(I)	42.77	0.17	858.43
	300(II)	43.33	0.30	1033.78
	400	45.11	0.48	1285.94
HDPE	300	28.91	0.19	1206.75
	400	27.91	2.06	970.78

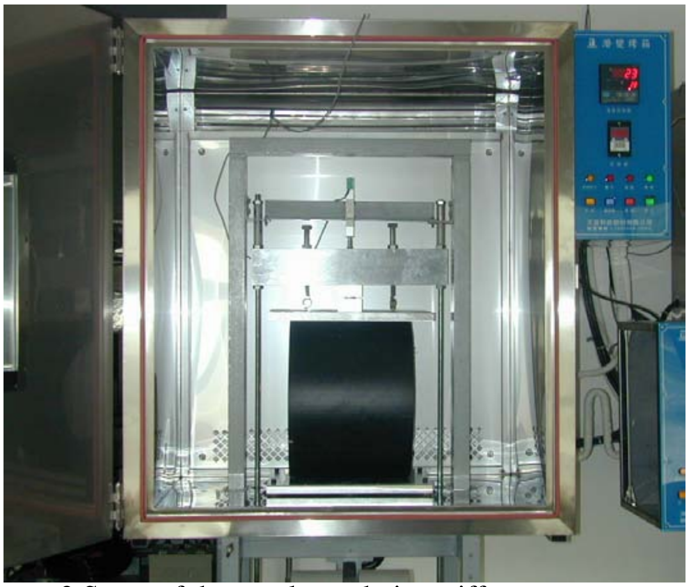

Figure 2 Setup of the accelerated pipe stiffness test apparatus.

Long-Term Pipe Deflection

Since the test materials included HDPE, PVC and ABS pipes and the test pipe sizes consisted of 300 and 400 mm, deflection (non-dimension) and pipe stiffness (PS) calculated at 5.0% deflections were used for the presentation. Even the initials inside diameter and pipe stiffness are different for each type of pipe. However, the use of this term would make the presentation easier.

A series of tests were performed using different percentage of the average initial pipe stiffness (Initial PS) value determined according to the ASTM D2412 standard test method. The tests were started from high percentage of initial PS load, such as 90%, 85%, and 80%, etc. The vertical deformation of the pipes was monitored using a LVDT and the data was collected using an automatic data collection system. Each test was terminated as the vertical deformation reaching 5.0% deflection of the initial inside diameter of the pipe.

Typical pipe deflection curves for ABS 300 mm pipe under various loadings were shown in the Figure 3. Since the required time to reach 5.0% deflection was quite different for various percentage of initial PS loading, the test data are presented on a semi-log coordinate system. The test duration (horizontal axis) is presented in log scale. The required time to let the pipe reaching 5.0% deflection increased rapidly as decreasing the applied loading. The test performed using 80% initial PS loading is reaching 5.0% deflection at the time near 10,000 hours. S-type deflection curves are shown in the figure on a semi-log coordinate system.

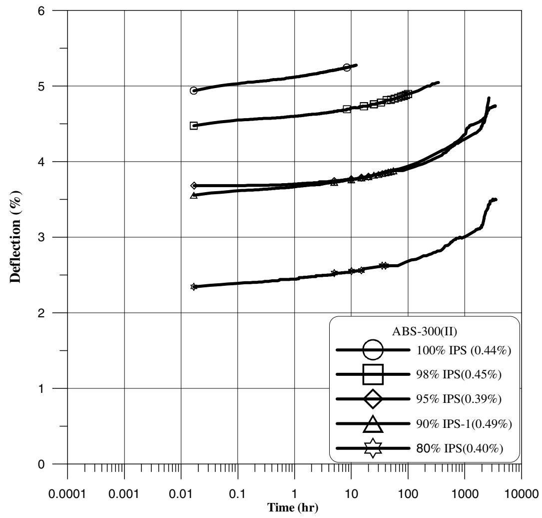

Figure 3 Long-term deflection curves for 300 mm ABS(II) plastic pipes.

The deflection curves under 80% of initial PS loading for HEPE, PVC, & ABS 300 & 400 mm pipes are shown in the Figure 4. In general, these curves are almost parallel to each other. For the test PVC pipes, the pipe stiffness values are about the same (around 2000 kPa). However, the curve associated with PVC 400 mm pipe shown greater deflection than that for 300 mm PVC pipe. Therefore, it is concluded that pipe deflection would increase almost proportional to the diameter of test pipe with similar initial PS value under parallel-plate loading mechanism. Based upon the PS values shown on the Table 2, the PS value for the test 300 mm ABS pipe (858 kPa) is about 2.43 times less than that for 300 mm PVC pipe (2092 kPa). The data shown on the Figure 4, the deflection associated with ABS pipe only shown slightly greater than that for PVC pipe. The difference of deflection is only about 20% between each other. The reason for causing this behavior required further study.

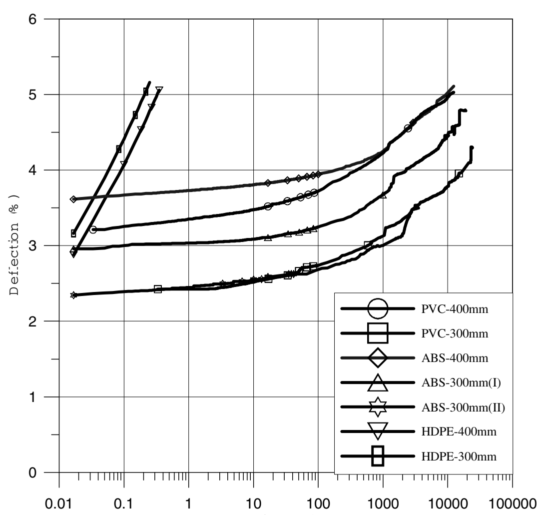

Figure 4 Long-term deflection curves for different types of plastic pipe under 80% initial PS loading.

Long-Term Pipe Stiffness

Based upon the data shown above, plastic pipe would continue to deflect under parallel-plate loading mechanism. However, the required time to reach 5.0% deflection would increase as decreasing of applied loading. Based upon this observation, it can be concluded that pipe stiffness would decrease as increasing the duration time under parallel-plate loading. The results of a series of tests for different pipe materials and diameters, the decreasing trend of PS value can be calculated for different deflection values. For the data observed in the study, the Figure 5 shown the PS values for the test pipes associated with 5.0% deflection. Even 300 and 400 mm PVC pipes consisted of similar initial pipe stiffness values, however, 400 mm PVC pipe deflected much faster than that for 300 mm PVC pipe under similar load. It is implied that, for the test pipes, the deflection of 400 mm PVC pipe could be a safety concern in comparing with 300 mm PVC pipe during service life. As shown in the figure, the PS value decreases as increasing the duration time on a semi-log scale for 400 mm and 300 mm HDPE, PVC and ABS pipes with 5.0% deflection. The observation of this behavior is still on going.

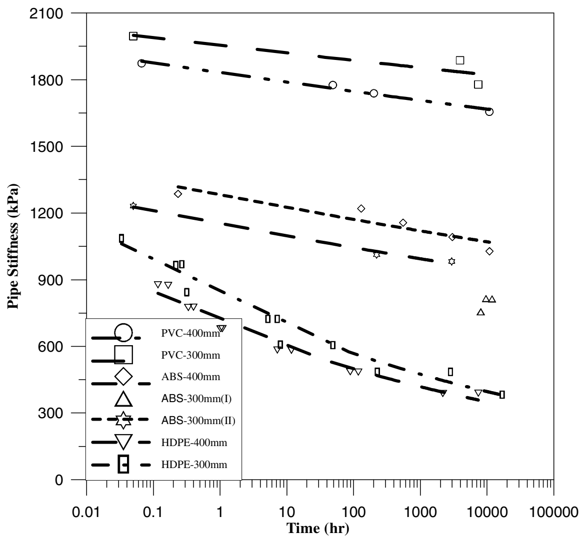

Figure 5 Long-term pipe stiffness values for different types of plastic pipes.

Accelerated Test Results

The elevated temperature process was used to accelerate the load deflection creep behavior in the study. Elevated temperatures of 40°C, 60°C, or 70°C were used in the test. At the present, only the accelerated tests of 300 mm and 400 mm HDPE pipes and 300 mm ABS pipes were completed. Typical time-deflection curves for 300 mm ABS pipes tested under 70°C are plotted in a semi-log scale shown in Figure 6. In addition, the pipe stiffness values can be computed based upon the accelerated test results. The computed pipe stiffness values versus duration plotted on a semi-log scale at different elevated temperatures for the 300 mm and 400 mm HDPE pipes and 300 mm ABS pipes are shown in Figures 7, 8, and 9 marking as the data points. The pipe stiffness values were further formulated on a log-log scale with parameters of duration time, test temperature, and pipe stiffness based upon current limited test data. The calculated pipe stiffness values based upon the developed equations (4) and (5) shown as the solid lines in the figures 7 and 8, respectively.

$$\mathrm{Log}\,(t) = -5.44 + 9388.75 \times T^{-1} - 0.98 \times \log\,(PS) \tag{4}$$
$$\mathrm{Log}\,(t) = -40.43 + 22046.40 \times T^{-1} - 3568.21 \times T^{-1} \times \log\,(PS) \tag{5}$$

where:

t = duration time (hour),
T= test temperature, and
PS = pipe stiffness.

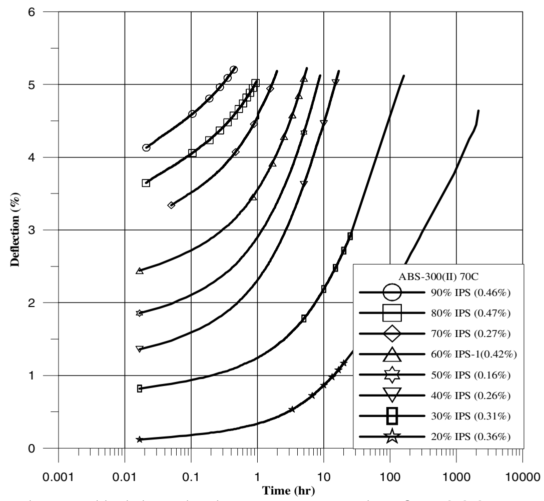

Figure 6 Accelerated parallel load plate test results for 300 mm ABS(II) pipes at 70°C elevated temperature.

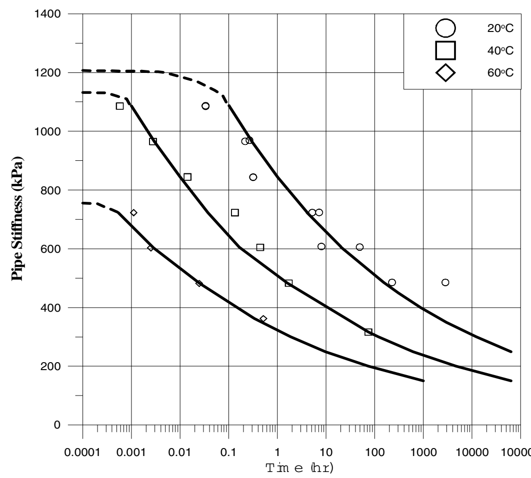

Figure 7 Pipe stiffness values for 300 mm HDPE tested at different elevated temperatures

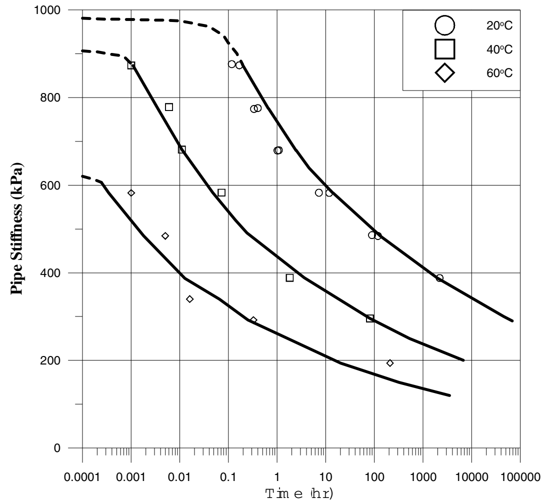

Figure 8 Pipe stiffness values for 400 mm HDPE tested at different elevated temperatures.

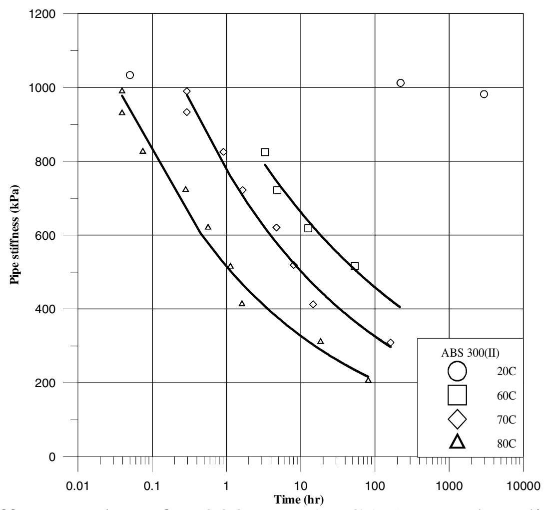

Figure 9 Pipe stiffness values for 300 mm ABS(II) tested at different elevated temperatures

Summary and Conclusions

The long-term deflection of HDPE, PVC and ABS plastic pipes under parallel-plate loading mechanism was investigated using conventional and accelerated test procedures. The nominal inside diameters of the test pipes were 300 and 400mm. The ASTM D2412 standard test method was used in the study. The following conclusions are made based on the previous discussions:

1. S-type long-term deflection curve was observed on a semi-log scale for HDPE, PVC and ABS plastic pipes under parallel-plate loading test.
2. 300 mm and 400 mm PVC test pipes consisted of similar initial pipe stiffness values, however, 400 mm PVC pipe shown almost 35% greater deflection in comparing with 300 mm PVC pipe under similar loading conditions. The

deflection rate approximately increases as increasing pipe diameter. However, the relationship between load-deflection and different plastic pipe materials is still unclear.
3. It would take around 10,000 hours to let 400 mm PVC and ABS plastic pipes reaching 5% deflection under conventional parallel-plate loading test, however, it seems that it would take very long time to let 300 mm PVC and ABS pipes reaching 5% deflection.
4. Long-term pipe deflection is a function of pipe material properties, pipe geometry, and loading conditions based upon the limited conventional parallel-plate loading tests. However, the deflection lag factor of Spangler's formulation should not be a constant value for estimating the pipe deflection.
5. Long-term pipe stiffness values decrease as increasing test duration on a semi-log scale for the test plastic pipes.
6. In comparison of the initial pipe stiffness values between 300 mm PVC and ABS pipes, the stiffness value of PVC pipe is about 2.43 times higher than that for ABS pipe. However, the deflection of ABS pipe is only about 20% higher than that for PVC pipe.
7. The use of higher test temperature for conducting parallel-plate loading test would accelerate the load-deflection response for plastic pipes and reduce the test duration time.
8. The formulation for estimating long-term pipe stiffness value was developed based upon the accelerated test results at different elevated temperatures for HDPE pipes. The use of this procedure for PVC and ABS pipes is still under investigated.

Acknowledgements

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Reference

ASTM D2412 (2002), "Standard Test Method for Determination of External Loading Characteristics of Plastic Pipe by Parallel-Plate Loading," American Society for Testing and Materials, West Conshohocken, Pennsylvania, USA.

Koerner, R.M., Halse, Y-H, and Lord, A. E., Jr. (1990), "Long-Term Durability and Aging of Geomembranes," *Proceeding of Waste containment Systems Geotech.* Special Publication No.26, ASCE, ed. R. Bonaparte, Philadelphia, PA: ASCE, pp. 106-134.

Koerner, R.M. (1998), *Designing with Geosynthetics*, 4th edition, Prentice-Hall, Inc., USA.

Moser, A.P. (1990), *Buried Pipe Design,* McGraw-Hill, Inc., USA.

Rinker Materials Corporation Homepage (2004), "PVC Sanitary Sewer Continues to Deflect, a 13-Year Study," www.rinkermaterials.com/hydroconduit/inforbriefs/i2007.htm.

Leak Detection on Wastewater Forcemains and Siphons in North America using the Sahara® Acoustic System

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A series of pilot studies across North America have been held to demonstrate that the Sahara® leak detection system can be adapted to detect leaks in pressurized wastewater forcemains and siphons under typical North American operating conditions. Likewise, the Water Research Council (WRc) in the UK has demonstrated the applicability of the Sahara® leak detection system to similar environments in the UK. Sahara® wastewater is a new technology that allows utilities to assess the condition of critical wastewater forcemains and siphons, such as major non-redundant lines, waterway crossings, and lines through environmentally sensitive areas, while keeping the line in service; it is the first to allow full length inspections of in-service force mains.

This paper discusses the need for inspections of wastewater forcemains and siphons, the Saraha® wastewater acoustic system, and the need for benchmarking of new technologies by third party organizations. The application of the Sahara system for the inspection of six wastewater pipelines in North America will also be discussed in detail.

Introduction

Forcemains, also known as rising mains in the United Kingdom, are pipelines in which wastewater is pumped under pressure. Forcemains are often used to transport waste from a pump station (lift station) that pumps the sewage up to a higher level so that sewage can begin to flow again by gravity to a trunk main. In some low lying areas sewage is collected by vacuum or low pressure systems and then is conveyed via forcemains. Siphons are also often used to transport waste across and under waterways.

Due to the high operation and maintenance cost of pumping the waste, the use of forcemains are primarily limited to locations where the design of a gravity flow pipeline systems is not feasible. Unlike gravity systems forcemains typically flow full. In North America it is estimated that approximately 60 percent of the forcemains are less than 12in (300mm) in diameter and flow velocities are normally low but can reach as high as 5 ft/s (1.5m/s) in the smaller diameter pipelines (4in or 100mm). Pressures are typically limited to 15 psi (100kPa) but can be as high as 150psi (1000kPa).

A variety of pipe materials can be used to construct forcemains (steel, iron, concrete pressure pipe, high density polyethylene (HDPE) and polyvinyl chloride (PVC). Most of the older forcemains were constructed using metallic materials that are susceptible to internal and external corrosion.

Failures of these systems can lead to loss of property value, public health hazards related to the risk of water contamination (e.g., fecal coliform bacteria), restriction of recreation activities in receiving waters and significant inconvenience to the public as many of these pipelines have no redundancy. The impact of raw sewage improperly discharged in environmentally sensitive areas, such as wetlands and rives, due to pipeline failures can be catastrophic to delicate aquatic ecosystems.

In March 2006 the failure of a 42-inch main in Waikiki, Hawaii resulted in the total of 48 million gallons of raw sewage to flow into the Ala Wai Canal during the six-day spill. This resulted in the mayor urging residents and visitors to heed signs warning against swimming and fishing in waters with high bacteria counts. Not letting the sewage to flow would have resulted in homes, businesses, restaurants and hotels with waste water backed up into those areas.

Over the last 20 years inspection technologies and protocols for gravity systems have been well developed and adopted around the world. However, the same can not be said for pressurized systems. The challenge for the inspection of pressurized systems is that most inspection technologies require the line to be taken out of service. Often this is not possible due to the lack of system redundancy. System surveys are also very expensive, difficult to complete, and data interpretation is subjective. To date the performance of forcemains in North America has been remarkable with few known or reported failures. However, as these systems continue to age and deteriorate failures will without a doubt increase in the near future. New and cost effective forcemain inspection technologies are a must if catastrophic and environmentally divesting failures are to be prevented.

Sahara® Wastewater is a new technology that allows utilities to assess the condition of critical wastewater forcemains and siphons while keeping them in service, and the first such service for forcemains.

The development, testing, implementation and adoption of a new technology can take years. The formation of third party technology benchmarking programs can reduce the time for technology development, testing and market implementation. For this reason the Pressure Pipe Inspection Company (PPIC) has teamed up with the Centre for Advancement of Trenchless Technologies (CATT) to conduct a pilot project to validate the suitability of Sahara® Wastewater at various locations across North America.

The remainder of this paper describes the Saraha® Wastewater acoustic system and the need for benchmarking of new technologies by third party organizations. The application of the Sahara system for the inspection of six wastewater pipelines in North America will also be discussed.

The Sahara® Wastewater Acoustic System

The Sahara® system is an inline condition assessment technology used throughout the world to detect and locate leaks in large diameter water pipelines 4 in (100 mm) or greater in diameter This section discusses the development history, operating parameters and components of the Sahara Wastewater system.

Development History

Sahara was developed in 1996 by the Water Research Center (WRc) in the UK as an alternative to leak noise correlators for water mains. It was designed for use in situations where correlators are ineffective: large diameter pipes (12in (300mm) or greater), non-metallic pipes, and pipes lacking the closely-spaced access points that correlators require. After several years of successful use in the UK, WRc licensed Sahara to The Pressure Pipe Inspection Company for North American distribution in 2004. Sahara has now been used for over 1,000 surveys on over 500 miles of pipelines, with over 1,000 leaks identified with an accuracy rate of over 99%. The history and operating parameters of the Sahara system for potable water are well discussed in the literature (Bond, 2001).

Sahara is a major step forward from the current state of the art, in that it can locate even very small leaks on pipes of any construction type. While hydraulic integrity does not in and of itself represent full condition assessment, it does confirm the structural integrity of the pipe, as defined by the EPA "the soundness of the pipe wall and joints for conveying water to its intended locations and preventing egress of water, loss of pressure, and entry of contaminants" (Royer, 2005). The EPA further confirms the value of locating small leaks in preventing pipe failure, when it notes "To the extent that main leaks can serve as a reliable indication of the location and timing of future main breaks, the detection, location, and quantification of main leaks is relevant to pre-failure detection, location, and prevention of main breaks." (Royer, 2005) As an example, all of the known failure modes for cast iron pipe (Makar et al, 2001) can cause leaks before they cause breaks, making leakage information a valuable component of condition assessment.

In 2005, WRc conducted eleven trials of a Sahara system adapted for use in wastewater lines in the UK. There were three significant differences from potable water that required consideration: the presence of solids and fats in the water, the different flow and pressure conditions, and the health and safety concerns regarding operation in wastewater. Adaptations were made to the propulsion system, the mechanism for insertion to operating pipes, and the operating procedures to accommodate these different conditions. The eleven trials conducted in the UK in 2005 demonstrated that the adapted Sahara system can indeed be used to conduct acoustic surveys of force mains and siphons in the UK (Jones et al, 2006).

In 2006, WRc sent the Sahara Wastewater prototype to North America for a 3 month pilot study. The objectives of the pilot were to confirm that the Sahara system can be operated in typical North American wastewater operating conditions, and to further study the operating parameters established for Sahara Wastewater.

Operating Parameters

The system parameters for Sahara Wastewater are based on the trials conducted in the UK. The purpose of the North American pilot is to verify these values in typical North American wastewater environments. These parameters suggest that Sahara Wastewater can operate in most wastewater forcemains and inverted siphons. The lines must have sufficient flow speeds to pull the sensor through the line, and must have sufficient pressure to ensure that any water escaping from the pipe generates the acoustic signature recognized by the system. Table 1 outlines system operating parameters required for a Sahara Wastewater survey.

Table 1: System operating parameters required for Sahara® Wastewater inline surveys

System Parameter	**Force main**	**Inverted siphon**
Pipeline Diameters	4" (100mm) or greater	12" (300mm) or greater
Flow Velocity	Minimum 3 ft/s (1 m/s)	Minimum 1 ft/s (0.3 m/s)
Single Survey Distance	Up to 2,500 ft (762 m)	Up to 5,000 ft (1,524 m)
Insertion Point	Any tap 3" (75mm) or greater	Siphon inlet chamber
Pump Duty Cycle	9 minutes / hour (15%) and up	N/A
Flow Velocity	Minimum 3 ft/s (1 m/s)	Minimum 1 ft/s (0.3 m/s)
Pressures in Survey Region	10 PSI (0.7 bars) to 150 PSI (10 bars)	
Cumulative Bends in Pipeline	Concrete or mortar lined pipes: up to 135 degrees Metallic and plastic pipes: up to 270 degrees	
Leak Sensitivity	As small as 1 Gallon/hr (1 L/hr) (depending on pressure)	
Pipeline Material	All Types	
Depth for precise tracking	Up to 33 ft (10m)	

Components

The system is made up of four major subsystems: the acoustic sensor and processing unit, insertion mechanism, sensor control system, and surface tracking tool (Figure 1).

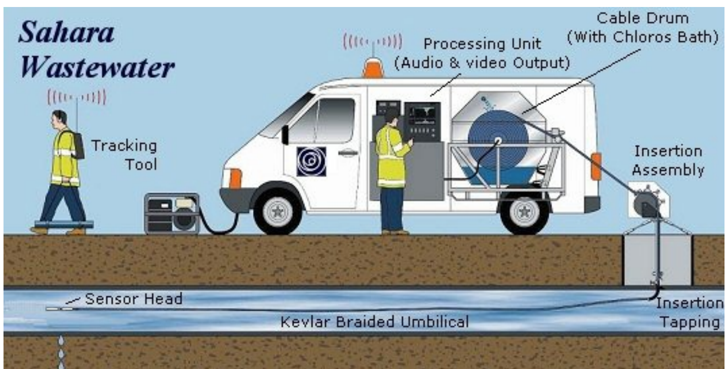

Figure 1. The Sahara® Wastewater acoustic system and its four subsystems.

In-line operation means that the Sahara acoustic sensor passes within one pipe diameter of any leaks, ensuring that the sound of even small leaks is picked up by the sensor. As the system travels through the pipe, the sensitive acoustic detector continuously “listens” for the distinctive noise of a leak made by the escape of under-pressure water. Leaks as small as 1L/hr (0.25 gallons/hr) are identified in real time by a processor at the insertion point (Bond, 2005).

Three different insertion mechanisms are available for insertion into wastewater lines. For forcemains, the standard Sahara insertion tube can be used, which allows for under pressure insertion into any tap with a diameter of 2in (50mm) or greater, so long as there are no bends along the 2in (50mm) insertion tap. Taps of 3in (75mm) diameter are preferred for wastewater surveys, to alleviate concerns that buildup of waste material on the sensor head could increase its diameter beyond 2in (50mm). A second *stub insertion assembly* is available for insertions at a pump station. The *stub insertion assembly* requires that the line be depressurized while the sensor is inserted into the line. This can often be achieved while the pumps are idle. Finally, a third insertion mechanism has been developed for use in siphon inlet structures. This system consists of a winch mounted siphon opening, and a guide tube to direct the sensor into the barrel of the line to be surveyed. All three insertion systems can be used on live, under-pressure pipelines.

The sensor and tether are pulled through the line by the flow of water. Several different propulsion mechanisms are available, depending on the insertion point, and the contents of the wastewater. The position of the sensor head is controlled in real-time using a hydraulically controlled tether. This gives the operator complete control of the system, allowing them to pass areas of interest multiple times, and to adapt to changing conditions as needed.

The PipeSpy 2000 surface tracking tool uses a powerful magnetic field generator to allow the surface location of the sensor to be determined. This allows for accurate location of any leaks or air pockets, or for mapping the pipeline location (Bond and Rees, 2001).

Benchmarking

The development, testing, implementation and adoption of a new technology can take years. The formation of third party technology benchmarking programs can reduce the time for technology development, testing and market implementation.

Key components of a technology benchmarking program are listed below in sequential order:

1. Evaluation of the technology to determine its potential to reduce buried infrastructure maintenance, operation, assessment, and/or rehabilitation costs.
2. Establishment of a group of industry partners committed to participate in the benchmarking study.
3. A third party organization, such as CATT, and industry partners establish the technology evaluation terms of reference.
4. Pilot testing the technology in each partner’s networks with third party data collection and analyses.

5. Upon completion of testing a partner's workshop to discuss the benefits and limitations of the technology.
6. Preparation of a preliminary report on the benchmarking program for each partner's review and comment.
7. A final partner's workshop to discuss the preliminary report so that a final consensus report can be issued.

For buried infrastructure condition assessment the technology must have the potential to reduce or optimize network replacement or renewal decisions and costs. If the technology can not meet this test then the usefulness of the technology should be questioned.

To test the technology a group of industry practitioners is required. The role of the industry practitioners include: assistance with the development of the technology evaluation terms of reference; providing funding to complete the surveys and project management, assignment of a project co-coordinator; selection and co-ordination of network surveys; assistance with data collection and analysis; and participation in technology evaluation workshops.

The role of the third party organization is to co-ordinate the project; assist with the development of the technology evaluation criteria; collection and analysis of survey data; preparation of preliminary and final reports.

We believe the formation of a technology benchmarking program is a win-win-win for the technology provider, industry practitioners and third party organizations respectively for the following reasons. The technology provider will receive direct industry feed back with respect to their expectations and needs, reduced marketing and sales costs, cost recovery for technology testing, a third party and industry vetted report that validates the technology. Main benefits for industry practitioners include: reduced cost and risk to evaluate a new technology; participation in the technology evaluation criteria; technology providers who develop a better understanding of industry needs and wants; shared knowledge and experience from all participants; and a third party technology evaluation report. For the third party technology provider the program is a win as they receive project funding that can be used to support organization needs, gain access to data and the development of network partnerships for future programs.

To establish a group of industry partners to participate in the Sahara Wastewater benchmarking initiative a meeting was held at the University of Waterloo on May 26, 2006 with CATT, PPIC and groups of interested industry practitioners. The meeting was held to determine if the interested parties had suitable pipes for field surveys and to set up a potential survey schedule for July to September 2006 using equipment on loan from WRc UK. Unfortunately due to the busy summer time frame, resources, the short time frame to find suitable pipelines for preliminary inspection, and budget constraints only two partners were able to conduct field surveys in 2006 - the City of Calgary and another City who shall be discussed anonymously. Results from these surveys are presented in the next section.

Case Studies

The City of Calgary

The City of Calgary is home to two major waterways: the Bow River and the Elbow River. Both rivers provide drinking water, fishing, and recreational opportunities, as well as enhance the beauty of the city. Keeping these rivers clean is a priority for the City of Calgary. One aspect of this is ensuring no wastewater enters them from pipelines underneath the rivers or their floodplains. The City of Calgary sees Sahara Wastewater as a tool to help prevent leakage from inverted siphons and force mains in these regions, and to reduce the likelihood of failures of these pipelines.

As such, the City of Calgary chose to participate in the Sahara Wastewater North American pilot to evaluate the Sahara Wastewater system. The City identified 13 candidate pipelines, and evaluated which would best serve to validate the viability of using the Sahara technology in wastewater environments, while posing the least risk of complications. Two inverted siphons and one force main were selected for surveys, with a planned total survey distance of approximately 4,756 feet (1450m).

On August 28-31, 2006, as part of the Sahara Wastewater North American Pilot, the Pressure Pipe Inspection Company (PPIC) inspected portions of the pipelines selected by the City of Calgary. The smooth operation of the project allowed the surveys to exceed the proposed scope without affecting the budget, both by exceeding anticipated survey distances, and by surveying more siphons than anticipated. A summary of results from the completed surveys can be found in Table 2.

Table 2. Results from the City of Calgary Sahara Wasterwater surveys

Survey Location	Planned Distance	Actual Distance	Flow Rate	Pressure	Total Bends
12” (300 mm) Steel Mackenzie Siphon	2,625 ft (800 m)	3,914 ft (1193 m)	3.6 f/s (1.1 m/s)	15 PSI (1.0 bars)	55º
28” (700 mm) Steel Nose Creek Siphon	130’ (40 m)	256’ (78 m)	5.5 f/s (1.7 m/s)	4.3 PSI (0.3 bars)	180º
12” (300 mm) AC Palliser Force Main	2,001’ (610 m)	1,995’ (608 m)	4.5 f/s (1.4 m/s)	50 PSI (3.4 bars)	105º
12” (300 mm) Steel Edworthy Siphon	-------	446’ (136 m)	Unknown	Unknown	180º

Standard deployment of the Sahara leak detection system was achieved in the 12in (300mm) Mackenzie Siphon. The proposal called for a survey of the first 2,500 ft (762m) of this siphon, as the Sahara Wastewater cables are limited to 2,625 ft (800m) length. The development of a connector capable of joining two such lengths of cable while withstanding a strain of up to 1,000 pounds (9.8N) allowed the survey to reach a distance of 3,914 ft (1193m).

Standard deployment of the Sahara leak detection system was also achieved in the 28in (700mm) Nose Creek Siphon. Significant electrical noise was found in the acoustic signal during the survey due to the presence of a nearby electrical transformer station. The survey was repeated with a new acoustic sensor designed to reduce electrical noise. This sensor successfully eliminated the electrical noise, allowing the successful completion of the survey. The full 256ft (78m) length of the siphon was surveyed.

Deployment of the Sahara leak detection system in the 12” (300mm) Palliser AC Force Main was possible when the pump in the Palliser Lift Station was turned on. The automatic pump cycle consisted of approximately 2.5 minutes of operation followed by 18 minutes standby. During pump operation the sensor deployed at a rate of approximately 160 ft/min (50m/min). The sensor was deployed as quickly as possible, and the pipeline was surveyed during the pullback at much lower speeds. The full 1,995 ft (608m) length of the force main was surveyed. A gas pocket was located approximately 260 ft (80m) into the survey, indicating a location at risk of deterioration due to hydrogen sulphide gas build-up.

Wooden supports were constructed to support the winch over the large manhole at the upstream end of the 12” (300mm) Edworthy Siphon. These allowed for the successful deployment of the Sahara leak detection system, and the full 446 ft (136m) length of the siphon was surveyed.

Case Study 2

This project involved the survey of five inverted siphons. Two are parallel barrels of a critical wastewater line. The remaining three are parallel barrels of an additional line of concern. A summary of results from the completed surveys can be found in Table 3.

Table 3. Results from the Sahara Wastewater surveys conducted on five inverted siphons.

Survey Identifier	Planned Distance	Actual Distance	Pressure	Total Bends
48” (1200 mm) PCCP Siphon	2,625’ (800 m)	2,920’ (890 m)	19 PSI (1.3 bars)	84º
54” (1370 mm) PCCP Siphon	2,625’ (800 m)	31’ (9 m)	19 PSI (1.3 bars)	84º
36” (900 mm) Ductile Iron Siphon	120’ (37 m)	150’ (45 m)	8 PSI (0.6 bars)	90º
30” (750 mm) Ductile Iron Siphon	120’ (37 m)	150’ (45 m)	8 PSI (0.6 bars)	90º
42” (1050 mm) Ductile Iron Siphon	120’ (37 m)	110’ (34 m)	8 PSI (0.6 bars)	90º

The 48in (1200mm) and 54in (1370mm) PCCP Siphons are two parallel siphons starting in a common inlet structure and ending in a common outlet structure. The siphons are part of a critical pipeline, and run through a sensitive area, making knowledge of their condition important to the operator. The utility noted the likely presence of debris inside this line, raising concerns over the possibility of snagging the cable.

The survey of the 48in (1200mm) line proceeded smoothly. The same connector used in Calgary was once again used to reach a distance further than anticipated. No leaks were detected.

Deployment into the 54in (1370mm) siphon could not be achieved beyond 31 ft (9.5m) from the inlet structure, which was used as the insertion point into the siphon. Several propulsion devices were employed, including drag chutes of different sizes, floaters, and combinations of the two. In all cases, the sensor was unable to deploy beyond 31 ft (9.5m) from the inlet structure. Investigation of the flow conditions at the inlet and outlet structures of the two parallel siphons revealed that the bulk of the flow entered through the 48in (1200mm) siphon, leaving the 54in (1370mm) siphon partially filled for the first several hundred feet from the insertion site. This, combined with the likely presence of debris in the line, and the consistency of the distance reached, suggests that the sensor was encountering a blockage of debris in the line.

The remaining three surveys (the 42in (1050mm), 36in (900mm), and 30in (750mm) Ductile Iron Siphons) were completed without complications. In two of the three siphons, the sensor was able to climb steep slopes on the downstream end of the siphon, allowing it to reach a greater distance than initially anticipated.

Next Steps

Based on the lessons learned and experience gained in the 2006 surveys the development of the benchmarking concept will be investigated in 2007. Further trails are planned for the summer of 2007, where participants shall be strongly encouraged to simulate leaks on the lines in question, so that the sensitivity of Sahara in wastewater environments can be reliably confirmed.

Conclusions

Sahara Wastewater is a new technology that allows utilities to assess the condition of critical wastewater forcemains and siphons, while keeping the line in service, and is the first technology to allow continuous assessments of forcemains. Most of the system operating parameters have been confirmed. Further testing is required to determine the system's sensitivity in wastewater environments.

Benchmarking of new technologies provides great value both to the developers and the users by reducing the time for development, testing, and market implementation. Due to the strict timelines of the equipment availability, only a small number of benchmarking tests were possible for Sahara Wastewater.

The Sahara Wastewater system was successfully piloted in two Cities in North America completing eight surveys in total. In all surveys, deployment of the Sahara Wastewater system occurred easily, and those few complications that were encountered were solved. However, no leaks were anticipated or identified in any of the surveys. Thus, the sensitivity of the device could not be confirmed. A gas pocket was found during the Palliser Lift Station survey in Calgary, indicating a location at risk of deterioration due to hydrogen sulphide gas build-up. Further testing and validation is required to determine the range of the Sahara Wastewater operating parameters and leak rate sensitivity.

References

Jones, C, Mergelas, B, Laven, K, Bond, A (2006). "Pinpointing Exfiltration in Large Diameter Pressurized Wastewater Pipelines with the Sahara Tethered System" Proceedings of ASCE Pipelines 2006, Chicago, Illinois, USA

Bond, A. and Rees, W. (2001). "Development of On-Line Inspection Systems for Water Mains", 4th International Conference on Water Pipeline Systems, BHR Group, York, 2001;

Michael D. Royer (2005). "EPA White Paper on Improvement of Structural Integrity Monitoring for Drinking Water Mains" Accessed December 14, 2006 from <http://www.epa.gov/ORD/NRMRL/pubs/600r05038/600r05038.pdf>

Makar, J.M., Desnoyers, R., McDonald, S.E. (2001). "Failure modes and mechanisms in gray cast iron pipe", Underground Infrastructure Research: Municipal, Industrial and Environmental Applications, Proceedings, Kitchener, Ontario, June 10-13, 2001, pp. 1-10

Surge Protection of a Wellfield Pipeline System Through Hardening and Risk Analysis

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Abstract

Utilities have several options for protecting pipeline systems from the effects of transients and surges. The most common methods include installation of surge protection devices, such as surge chambers, air release valves, etc., and to design the system to withstand the stresses of transients (hardening of the system).

The San Antonio Water System recently placed into operation an Aquifer Storage and Recovery (ASR) wellfield that both stores excess water underground (storage mode) and pumps stored water and native groundwater (recovery mode) for use in its distribution system. The system is unique in that it must operate in both directions. In the storage mode, the wellfield is connected directly to the distribution system and must be designed to withstand the tremendous pressure associated with receiving water from a pressure zone over 350 feet (107 m) in elevation above the wellfield's ground elevation. In recovery mode the wells pump to a treatment plant or ground storage tank at approximately 50 feet (15 m) higher in elevation. The high pressure experienced during storage operations and the low pressure experienced during recovery operations each have their own unique transient conditions.

The 32-well wellfield will be constructed in three phases ultimately producing 80 million gallons per day (mgd) (303,000 m^3/day), with the second phase currently in design and construction and the third phase in conceptual design. The first phase of the wellfield (30 mgd (114,000 m^3/day) capacity) was constructed with PVC C900 and C905 for the laterals and small collection mains, and AWWA C200 mortar-lined steel pipe for the mains greater than 24 inches (61 cm) in diameter. The surge protection system included four surge chambers located on the main line designed to keep the wellfield under positive pressure during surge events.

A surge analysis was performed prior to design of the second phase of the wellfield and included a concurrent analysis of the third phase surge requirements. The surge analysis recommended the installation of additional surge chambers, and to save costs, the allowable minimal pressure would be set at –7.0 pounds per square inch (psi) (-0.48 bar) throughout the wellfield. The cost to fully protect the wellfield exceeded $1 million for these options. In an effort to reduce costs further, the utility decided to harden the second and third phases of the system so that they could withstand pressures below –7.0 psi (-0.48 bar). The second and third phase piping only included laterals that would connect to the existing C905 and C200 mains. Therefore, HDPE pipe was selected for the second and third phase piping because it could withstand transient pressures down to full vacuum (-14.7 psi) (-1.0 bar). In lieu

of using surge chambers on the new piping, the utility decided to use air release valves to maintain a pressure above –10.0 psi (-0.69 bar) in the HDPE piping. The cost to provide this level of protection approached $500,000.

In an effort to further reduce costs, the utility applied a risk analysis to the wellfield piping and adopted a "wait and see" policy before investing significant resources into the surge protection system. The reasons for this decision are 1) the wellfield is not a critical water supply component and can be taken offline for repairs, 2) the expensive components of the wellfield, i.e., the wells and the C200 mains, are well protected with surge protection devices installed during Phase 1, and 3) with proper monitoring, the utility can detect and repair transient-damaged sections less expensively than to blanket the entire wellfield with surge protection.

Background

In 1997, the San Antonio Water System (SAWS) embarked upon a study to determine the feasibility of developing an aquifer storage recovery (ASR) facility to provide potable water to meet peak seasonal demands and reduce peak withdrawal rates from the Edwards aquifer. The test drilling program suggested that the Carrizo aquifer was suitable for storage of several potential source waters. Also, during the second part of the feasibility investigation, it was determined that the Carrizo aquifer in South Bexar County could provide a substantial supply of new-source groundwater for many years.

Therefore, SAWS determined that the development of a conjunctive use project, incorporating an ASR element and a groundwater production element, would be in the interest of San Antonio. This approach provides the most rapid development of the supply source. During the first phase, the ASR system was developed to provide a groundwater supply of 30 mgd (114,000 m^3/day). A total of 17 wells were constructed to supply raw groundwater to a new water treatment plant, pumping station, and transmission pipeline. A bypass around the pumping station and the water treatment plant was constructed to facilitate recharge.

Water from these facilities is transported into San Antonio to three terminal points where it is delivered into the SAWS distribution system. Currently, these points are the Seale Pump Station, the Artesia Pump Station, and the Randolph Pump Station.

Phase 2 of the ASR Program originally anticipated expansion of the system to 60 mgd (227,000 m^3/day) of recovery capacity. All Phase 1 facilities were designed to operate at a minimum 60-mgd (227,000 m^3/day) rate. Enhancements necessary to achieve the 60-mgd (227,000 m^3/day) capacity would include construction of up to 17 new ASR wells and the installation of one 20-mgd (76,000 m^3/day) high service pump at the water treatment plant high service pump station (HSPS). A third phase would increase the capacity of the wellfield to 80 mgd (303,000 m^3/day). The layout of the wellfield is shown in Figure 1. This layout shows all existing and projected well sites as well as existing surge chamber locations.

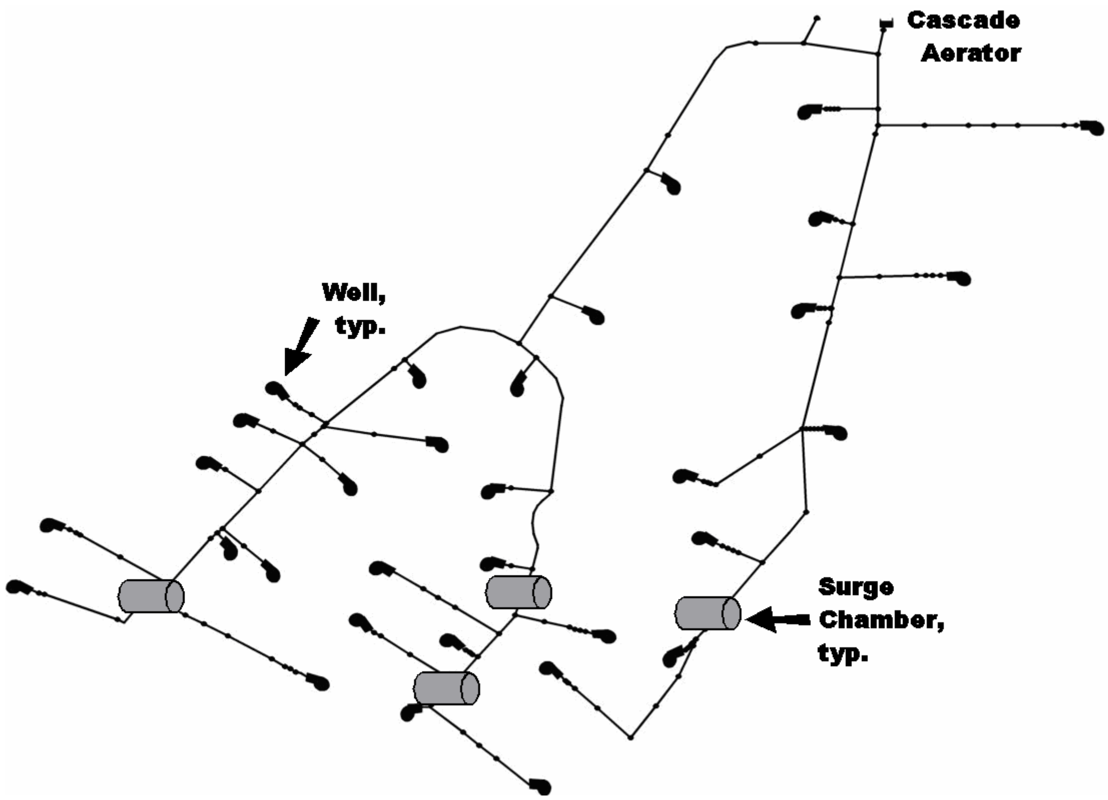

Figure 1. Layout of ASR Wellfield

Table 1. PVC Pipe Characteristics

Pipe Class	OD, in. (cm)	Type	Wall thickness, in. (cm)	DR	Long Term Strength, psi (bar)
100	12 (30)	C900	0.48 (1.2)	25	4,000 (276)
165	20 (51	C905	0.80 (2.0)	25	4,000 (276)
165	24 (61)	C905	0.96 (2.4)	25	4,000(276)

The Phase 1 wellfield piping consists of C200 mortar-lined, tape coated steel pipe and C900 and C905 polyvinyl chloride (PVC) pipe. The size and characteristics of the PVC pipe is shown in Table 1 and the size and characteristics of the steel pipe is shown in Table 2. All of the main piping 24 inches (61 cm) in diameter and larger was constructed of steel pipe and all laterals were PVC. During Phase 1, four 300-ft^3 (8.5 m^3) surge chambers were strategically located throughout the wellfield to protect the Phase 1 and Phase 2 piping from surge pressures. The initial air volume of each surge chamber was set at 50%.

Table 2. Steel Pipe Characteristics

Pipe Class	ID, in. (cm)	OD of Steel barrel, in. (cm)	Wall thickness, in. (cm)	Specified Tensile Strength of Steel, psi (bar)	Specified Yield Stress of Steel, psi (bar)
150	30 (76)	31.5 (80)	0.25 (0.64)	60,000 (4,100)	42,000 (2,900)
150	36 (91)	37.5 (95)	0.25 (0.64)	60,000 (4,100)	42,000 (2,900)
150	42 (107)	43.5 (110)	0.25 (0.64)	60,000 (4,100)	42,000 (2,900)
150	54 (137)	55.54 (141)	0.27 (0.69)	60,000 (4,100)	42,000 (2,900)
150	60 (152)	61.62 (157)	0.31 (0.79)	60,000 (4,100)	42,000 (2,900)

The wellfield operates in both directions. During recovery, water is pumped from the aquifer to the cascade aerator for treatment or into a storage tank near the HSPS. During storage, the water enters the wellfield through the bypass and is injected into wells for storage in the aquifer. For the purposes of this analysis, only the recovery mode of operation was analyzed because it presented unique operating conditions for the wellfield piping. During recharge, a pressure-reducing station will control the pressure within the wellfield which will reduce the effects of positive and negative surge events.

Surge Protection Options

During recovery operations, a surge analysis revealed that the wellfield would only be subjected to very minor positive surge pressures, all within the operating range of the infrastructure. However, because of the long lateral lengths and the low operating pressure throughout the wellfield during recover, the majority of the transient problems would occur as a result of negative pressures.

Several scenarios were modeled which resulted in a negative surge pressure within the wellfield. These scenarios included loss of power, valve closure, and line breakage. Of all these scenarios, loss of power during peak production resulted in the greatest negative pressure. Significant negative pressures can result in pipe collapse or joint movement. One additional concern was the possibility of the cement mortar lining spalling because of excess stress in the steel pipe walls and because accumulated moisture between the lining and the pipe wall can push the lining out as the line pressure becomes negative.

Several options were presented to SAWS to protect the steel and PVC pipe from the effects of the negative surge. These options include installing additional surge chambers, installing additional air release/vacuum valves, and increasing the strength of the infrastructure, or “hardening” of the system. Each of these systems have their benefits and negative attributes and a risk analysis must be performed before accepting one or more as the final solution.

Surge chambers have a well-established reputation for surge protection against both high and negative surge pressures. During Phase 1, four surge chambers were installed to protect the steel main lines. However, surge chambers require specialized control equipment, a power source, and a monitoring system in order to be dependable when they are needed. Another negative attribute of the surge chambers is their initial capital cost and constant maintenance requirements.

Air release and air vacuum valves also have a well-established reputation for protecting pipelines from the effects of surge. However, there are also several documented cases in which pipelines have been damaged beyond repair because of an undersized or under-maintained air vacuum valve.

A third option for protecting the wellfield from negative surge pressures is to harden the system. Hardening of the system includes installing pipelines and appurtenances that are designed to operate under negative pressure conditions (up to complete vacuum of -14.7 psi (-1.0 bar)) as well as the cavitational forces associated with column separation and the return of positive pressures.

One of the most expensive components of the wellfield is the piping. Therefore, it is important to consider choosing a piping material that can withstand negative pressures up to and including complete vacuum. High density polyethylene, or HDPE, pipe presented the characteristics necessary to operate under negative surge conditions. When properly installed in a trench, the pipe will not collapse under full vacuum conditions.

Operation and Protection of the Wellfield Piping

In order to operate the wellfield with a factor of safety of 2.0, the recommended lowest pressure at any point in the wellfield would be approximately -7.0 psi (-0.48 bar). However, in order to maintain this pressure, several surge chambers and air release/vacuum valves would have to be installed, resulting in a surge protection cost approaching $1 million. Even limiting pressures to -10.0 psi (-0.69 bar) would result in a surge protection cost of approximately $500,000. Faced with these costs, SAWS was presented with options to harden the wellfield system to reduce risk of damaging the piping during a surge event.

The use of HDPE pipe was selected as the option to protect all Phase 2 piping. However, the existing system had several miles of PVC laterals and steel main piping that required protection during a surge event. The surge analysis showed that the four existing surge chambers protected the majority of the steel piping from extreme negative pressures. Therefore, it appeared from these results that the risk remained with the PVC laterals.

SAWS had several options for protecting these existing lateral pipes. One option was to replace all of the pipes with HDPE pipe. This option was not selected because of the high cost of installing several miles of HDPE pipe and because it was deemed as wasteful. A second option was to install smaller surge chambers at each of the well sites. This option will protect the PVC laterals, however, the high costs associated with this option made it impractical.

The final option was to adopt a “wait and see” operational scenario. Based on literature of PVC pipe characteristics, it was difficult to determine if any real damage would result if the PVC pipe was subjected to a full vacuum. Laboratory tests had shown that the pipe wall could adequately withstand full vacuum pressure, but the gasket was not rated for that situation. Even though the gasket was not rated for vacuum, there was no data that showed that the gasket would fail under vacuum conditions.

Based upon this information, SAWS chose the option to leave the PVC laterals in place and closely monitor the condition of the pipe. If a power failure is experienced within the wellfield, SAWS has the option of isolating all of the PVC laterals and testing their integrity individually. This provides SAWS with the option of repairing any damaged pipe before the lateral is returned to service. The damaged section of pipe could also be replaced with HDPE pipe so as to minimize the possibility of future damage at the same location during future surge events.

Conclusion

Water supply systems are constantly performing risk assessments and cost/benefit analyses to determine how much budget to spend on protecting their water supply and delivery systems. The San Antonio Water System was provided with an option of providing limited additional protection for existing piping within the aquifer storage and recovery wellfield because of several unique factors associated with the wellfield. These factors include 1) the wellfield is not a critical water supply component and can be taken offline for repairs, 2) the expensive components of the wellfield, i.e., the wells and the C200 mains, are well protected with surge protection devices installed during Phase 1, and 3) with proper monitoring, the utility can detect and repair transient-damaged sections less expensively than to blanket the entire wellfield with surge protection.

A Basis for Using Single-Welded or Double-Welded Lap-Joints for Steel Water Pipe

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Abstract

Field-welded lap-joints were first successfully used in the 1920s to join sections of an all-steel pipe transmission line in Texas. This application was later improved using shielded metal arc-welding technology followed by other semiautomated arc-welding processes (i.e., flux-core arc-welding and gas-metal arc-welding) that currently dominate construction practices for field-welding water transmission lines. Pipeline designers agree that arc-welding processes provide a superior field-welded joint; however, there is considerable uncertainty regarding pipeline design loads and required field-welded joint details necessary to resist these loads. Field-welded lap-joints have proven to be very economical and are contractors' first choice; however, they are also the most vulnerable part of any welded lap-joint steel pipeline. There have been significant failures over the last 80 years since field-welded lap-joints were first introduced. With this in mind, it is unfortunate that design manuals fail to focus on this critical aspect of structural design.

The longitudinal forces on buried, continuous, steel water pipelines can be extremely large (e.g., thermal force, Poisson's stress, differential settlement, hydrostatic thrust, seismic force, and soil drag force if pipelines traverse steep terrain) and even more so for large-diameter pipelines. These load requirements are not covered well in the common design standards (AWWA, 2004) and can be hard to quantify. Many designers believe that these longitudinal forces are relatively small. The authors recently reviewed a pipeline design criteria report that stated continuous, buried, steel pipelines are not subject to longitudinal forces greater than hydrostatic thrust. When problems with buried pipe joints develop, fixing or repairing them is expensive and disruptive. Dewatering and accessing a large-diameter main is a major undertaking. Using double-welded lap joints costs more than single-lap welds, but it is better from a corrosion and structural standpoint and allows pressure-testing to ensure that joints are water tight. Further, there are two welds to resist leakage, which are much like the difference between single-gasket and double-gasket joints. It's often a sound investment for infrastructure that you want to serve without problems for many years. There is a place for single lap-welded joints, but they are not a panacea to be used everywhere. When the stakes are high, the designer should consider double lap-welding or even complete joint penetration (CJP) welding. This paper proposes methods for determining welded steel pipeline forces and strategies for resisting these forces with field-welded joints. The application of field-welded lap-joints that are single-welded is compared with double-welded lap-joints

Pipeline Forces

Welded steel pipelines that are internally restrained from longitudinal movement by field welded joints must be designed to resist longitudinal forces as follows:

1. **Thermal force:** A temperature change often occurs when pipe is installed at one temperature but must carry water at a different temperature. This temperature change (or delta T) causes a stress in buried pipeline that is restrained from longitudinal movement due to soil friction forces. The computation in Figure 1 demonstrates the stress that restrained steel will experience for every degree Fahrenheit (°F) of temperature change.

   For example, if a pipeline with a wall thickness measuring t = 0.50 inches, were buried at a temperature of 90°F and subsequently carries water at a temperature of 50° F, the delta T = (90 - 50) = 40° F. The longitudinal thermal stress in the pipe wall, assuming the pipe cannot move longitudinally would be 188.5 x 40 = 7540 pounds per square inch (psi) tension. The thermal unit force, per inch of pipe circumference, would be 188.5 x 40 x .50 = 3,770 lbs.

2. **Poisson's effect:** Poisson's effect causes a pipe to contract longitudinally as result of radial expansion due to internal pressure. In a buried pipeline that expands in radial direction but is restrained in the longitudinal direction, Poisson's effect results in a longitudinal tension stress. Poisson's ratio for steel is 0.30; the longitudinal stress resulting from Poisson's effect can be estimated by knowing the pipeline hoop stress. For example, if the hoop stress is 18,000 psi, then Poisson's effect results in a longitudinal stress of 0.3 x 18,000 = 5,400 psi.

3. **Differential settlement:** Differential settlement can occur where the pipeline is unevenly supported. Beam action results in bending and shear stresses in the pipeline. Forces resulting from differential settlement can become very large and can lead to pipeline rupture. For any particular beam bending curvature, the maximum bending stress increases linearly with pipe diameter; therefore, large-diameter pipe can be especially impacted by settlement. Some differential settlement—even if not anticipated—is likely to occur because of normal construction methods or a variation in trench backfill or native soils so the pipeline joint design must be robust enough to accommodate it.

$E := 29000000 \cdot psi$	Young's Modulus for steel
$\varepsilon = 6.5 \text{ x } 10^{-6} \frac{in}{in \cdot \Delta°F}$	Coefficient of thermal expansion for steel
$\sigma_\varepsilon := E \cdot \varepsilon$ | $\sigma_\varepsilon = 188.5 \frac{psi}{\Delta°F}$	Thermal stress per degree change F

Figure 1. Stress Computations for Restrained Steel.

4. **Hydrostatic thrust (PA force):** Internal pressure force of the fluid, including transient forces within a pipeline that acts at bends, tees, valves, reducers, and bulkheads, creates a hydrostatic thrust force that is the product of the pressure (P) and internal area (A); this is often referred to as a PA force.

5. **Seismic force:** Earthquakes can generate strong ground motion and pressure waves that affect pipeline design. Raleigh waves as the result of a seismic event can travel along pipelines at a high velocity. The forces from these waves can be magnified where pipelines enter underground vault structures for example and can rupture the pipe if the joint design is not adequate.

6. **Soil drag force (if pipelines traverse steep terrain):** When a pipeline is installed on an unstable steep slope, soil drag forces can be developed that will act on the pipeline, pulling it in the downhill direction. The soil drag forces are related to soil and pipe wall friction, soil density, and depth of soil cover over the pipe. Often, to prevent downhill movement of a pipeline located on a steep slope, anchor blocks are used. The pipeline transfers the drag forces to the anchor blocks so that the joint design must adequately resist the soil drag forces.

Pipeline Load Combinations

For buried pipe, an estimate of required longitudinal strength is necessary before selecting a joint type. Good pipeline design requires that all load combinations be considered and that the greatest load combination be selected for design.

Longitudinal hydrostatic stresses (from PA forces) are generated if the pipe is free to move longitudinally. Thermal and Poisson forces are generated when the pipe is restrained from longitudinal movement by soil friction forces or other external restraint. Since movement and non-movement cannot occur simultaneously, designers often consider each condition separately. PA forces are considered to be a separate loading condition from the combined effects of thermal forces plus Poisson's forces.

In most design references, the effects of special loading conditions—such as differential settlement, seismic loading, or soil drag forces—are considered separately and are seldom combined with other loading conditions, although some combinations with PA, thermal, and Poisson forces are certainly possible.

Welded Joint Strength

A good source of information regarding field welded joint strength can be found in the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessels (BPV) Code, Section VIII, Division 1, UW-12, which is outlined in Table 1 below in abbreviated form.

As indicated above in Table 1, butt-joint welds that are made with CJP welds and are radiographically tested (RT) and are assigned the highest strength levels. For example, a joint efficiency E = 1.0 (i.e., same strength as adjacent pipe wall) is given for a double-welded CJP butt-joint (no backing) with 100 percent RT. The same joint with no RT is assigned joint efficiency of only E = 0.70 (i.e., only 70 percent as strong as the adjacent pipe wall).

Table 1. Joint Efficiencies (ASME, 2004).

Type No.	Joint Description	(a) Full RT	(b) Spot RT	(c) None
1	Butt joints as attained by double-welding or by other means which will obtain the same quality of deposited weld metal on the inside and outside weld surfaces to agree with the requirements of UW-35. Welds using metal backing strips which remain in place are excluded.	1.00	0.85	0.70
4	Double full-fillet lap-joint	NA	NA	0.55
6	Single full-fillet lap-joint without plug welds	NA	NA	0.45
RT	radiographically tested			

Double and single full-fillet-welded lap-joints result in added bending and shear stress because of the eccentric load path. They cannot be RT inspected because of geometric limitations and are assigned joint efficiencies of only E = 0.55 and E = 0.45, respectively. An increase in E from 0.45 to 0.55 is a 22 percent strength increase. In the opinion of the authors, the actual strength increase is probably substantially greater than 22 percent; however, little meaningful testing is available to confirm this. It should also be noted that ASME BPV Code, Section VIII, Division 1, Table UW-12, limits fillet-welded lap-joints on pressure vessels to 5/8-inch maximum shell thickness.

Field-Welded Joints

Figure 2 illustrates common field-welded joints often used for pipeline work.

Welding Code Advice

The following limitation is applied to using single-fillet welds in lap-joints by the American Welding Society (AWS) D1.1, Structural Welding Code-Steel (AWS, 2006):

> *2.8.1 Lap Joints, 2.8.1.1 Transverse Fillet Welds. Transverse fillet welds in lap joints transferring stress between axially loaded parts shall be double-fillet welded except where deflection of the joint is sufficiently restrained to prevent opening under load*

Figure 3 illustrates the reason the AWS D1.1 Code imposes this limitation. The hoop strength of the pipe provides restraint against opening; however, it is arguable that the hoop restraint is sufficient to prevent opening.

The double fillet-welded lap-joint will also rotate under load (Figure 4); however, joint separation and tearing is limited by the restraining force provided by the second weld, giving this joint superior overload capability over that of a single fillet-welded lap-joint.

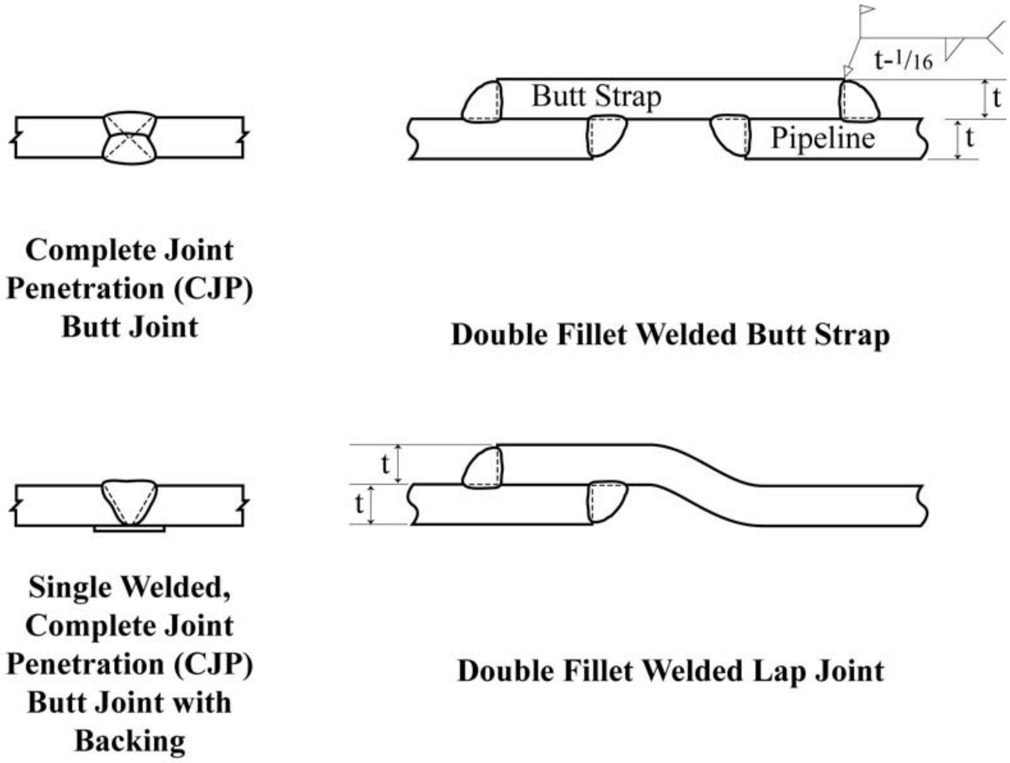

Figure 2. Common Field-Welded Joints.

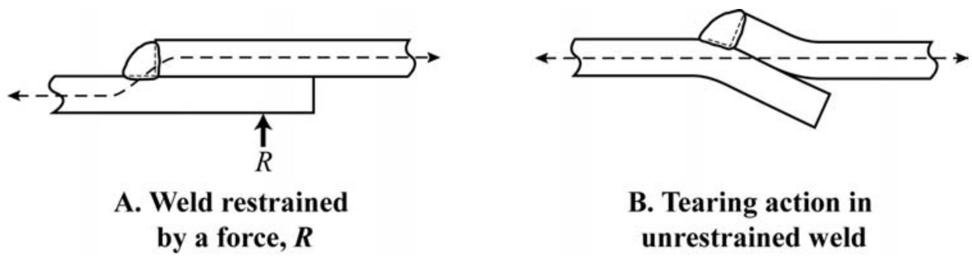

Figure 3. Single Fillet-Welded Lap Joints.

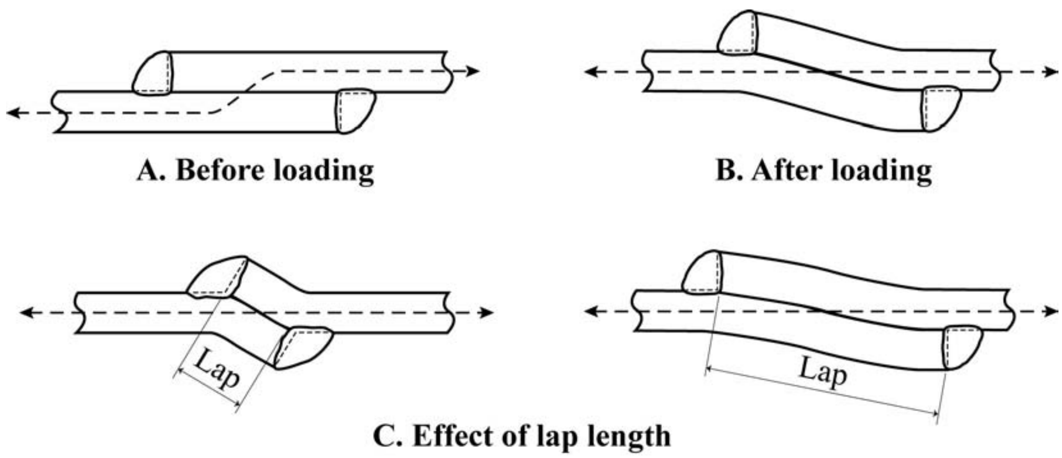

Figure 4. Double Fillet-Welded Lap Joints.

Comparison of Double- and Single-Welded Lap-Joints

The lap joint is preferred by contractors because it is much easier and faster to install than butt-joint welds and fit-up time is substantially less, which translates into cost savings. With an ideal cylindrical pressure vessel, statistics show that the longitudinal PA stress is only half of the hoop stress. Therefore, for such a pressure vessel minimum joint efficiency requirements for a girth weld to resist a PA force can never exceed E = 0.50. If ASME joint efficiencies are used, then a double fillet-welded lap-joint efficiency (E = 0.55) will satisfy this requirement and is frequently used for pipeline conveyance work. It should be noted, however, that a single fillet-welded lap-joint (ASME E = 0.45) causes the joint strength to control the design over the hoop stress, and an increase in shell thickness is necessary when comparing this joint with the double fillet-welded lap-joint. In pipeline application, the load combination of thermal force with Poisson's effect often governs over PA force, making the single-full fillet-welded lap-joint less desirable from a strength perspective.

Aside from the obvious structural advantages of the double full-welded lap-joint in protecting against unexpected stresses that often occur outside the common design, following are some other considerations as well:

- Double-welded, full-fillet lap-joints provide double protection over single welds against long-term leakage.
- Double-welded, full-fillet lap-joints allow for air-testing between the joint assuring a water-tight joint before backfilling.
- Double-welding protects the uncoated faying surfaces between the bell and spigot from moisture and potential corrosion.

And finally, the authors have three last thoughts:

- Some pipelines are more important than others. If the pipeline is a critical piece of infrastructure needing to serve without interruption, then the increased safety factor provided by the double-welded, full-fillet lap-joint is probably good insurance. Shutting down, dewatering, and mobilizing for repairs can be very costly and disruptive.
- Should the need occur to increase joint strength in the future because of seismic loading, pressure increases, or any other reason, then it will be extremely expensive to accomplish. The increased cost of double-welding during installation will be a fraction of the cost of doing it later. In fact, it might be virtually impossible to do it later.
- In terms of the total project cost, the difference in cost of single- versus double-welding is a relatively small percentage.

Recommendations

- Consider all potential combinations of longitudinal forces in buried welded-joint steel pipelines that can be caused by hydrostatic, thermal, and Poisson's effects; steep slopes; and differential settlement.

- Consider that, in addition to significant increased strength, double-welded lap-joints enhance corrosion and leak protection and allow for physical-joint leak-testing before the joint coating and lining and before trench backfilling and before filling the pipeline with water.
- Consider all factors, including the importance and expected service life of the pipeline in deciding on the type of joint to be used.
- Further meaningful, full-scale testing to accurately compare the strengths of single- versus double-welded lap joints is needed.

References

American Society of Mechanical Engineers (ASME). (2004). *Boiler and Pressure Vessels Code, Section VIII, Rules for Construction of Pressure Vessels, Division 1, UW-12*.

American Water Works Association (AWWA). (2004). *Steel Water Pipe: A Guide for Design and Installation (M11), Fourth Edition.*

American Welding Society (AWS). (2004). *Structural Welding Code – Steel D1.1/D1.1M-2006*.

When A Train Comes – You Must Move
Transit Rail Lines Impacting Underground Infrastructure

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In response to a voter referendum, the Metropolitan Transit Authority of Harris County (METRO) is committed to extending its highly successful initial 7 miles of Light Rail Transit system to more than 45 miles within a 4-year period. METRO's plans call for all 30 miles to be under construction simultaneously in 5 different corridors. As a result of METRO's aggressive schedule, the City of Houston has to relocate numerous critical underground facilities. These relocations include force mains, lift stations, and several large diameter water transmission mains including an extremely vital 66-inch water main.

A major focus of the paper will be to present the issues associated with the 66-inch PCCP water main. This line is the sole source of treated water to the City's Southwest Pump Station. This station averages more than 75 MGD and its service area extends over 80 square miles, including two major employment centers and the Houston Galleria, the largest retail center in the southwest. The discussion will include how five miles of this vital PCCP water main will be impacted by the Light Rail Transit extension. Impacts, analysis, and alternatives evaluated and to be discussed include how to mitigate the system's electrical currents, providing access to the line while reducing impacts to the rail operations, and how to minimize the potential failures on this line.

This paper will also present the difficulties caused by METRO's aggressive schedule on the City's critical infrastructure. It will present how the City is dealing with the coordination necessary for the construction activities occurring simultaneous in five different areas.

Background

In response to a voter referendum, the Metropolitan Transit Authority of Harris County (METRO) is committed to extending its highly successful initial 7 miles of Light Rail Transit (LRT) system to more than 45 miles within a 4-year period. This extension will cost between $1.5 and $2.0 billion, equally split between local and federal transit dollars.

METRO's authorization for the LRT extension came only after a bitterly fought election. This election was forced upon METRO, and as such, METRO feels compelled to adhere to the letter of the ballot language.

To obtain voter approval, one of METRO's strategies was to furnish LRT to areas with high projected ridership with the hope of "getting the vote out" in favor of the LRT extension. Consistent with the referendum, METRO's plans call for all 30 miles to be under construction simultaneously in 5 different corridors as shown in Figure 1 below.

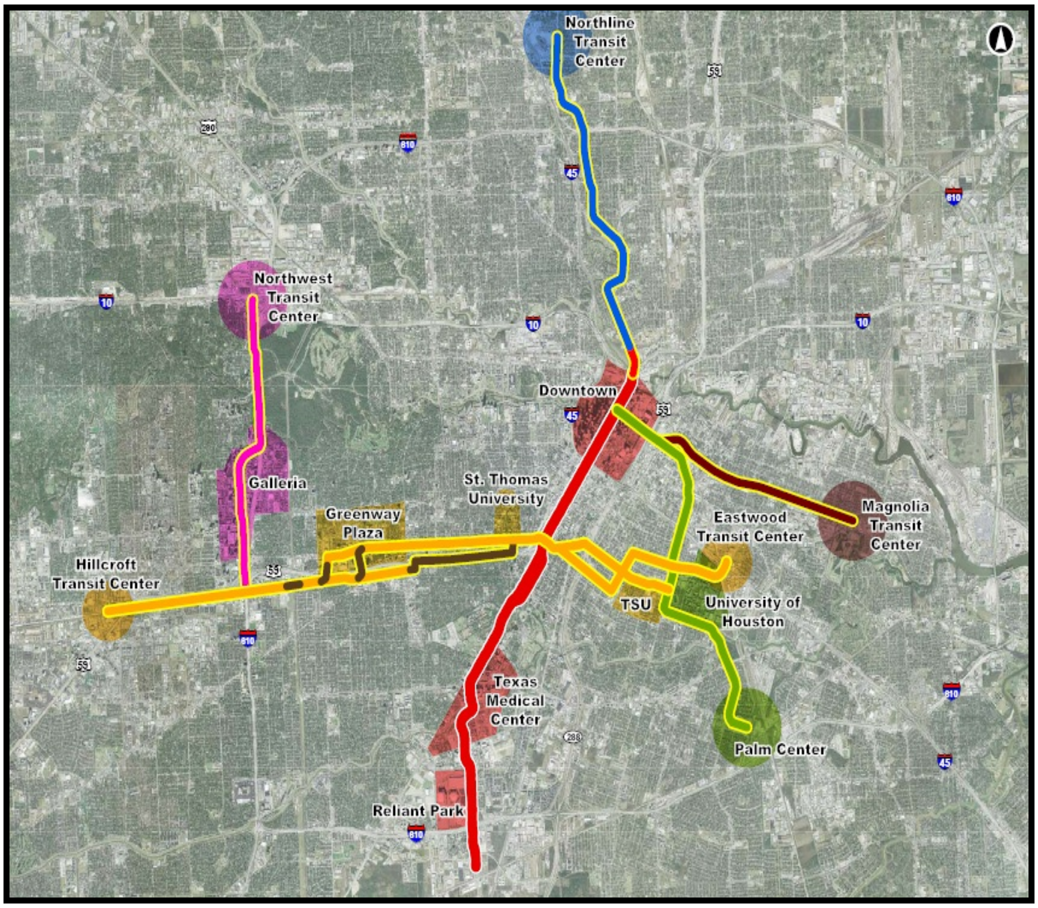

Figure 1. Five Corridors for METRO Solutions Plan

After over 12 months of a solicitation process, METRO selected a consortium to extend the 30 miles using a special Texas legislatively authorized process via a design, build, operate and maintain (DBOM). This process has specific requirements normally not seen across the nation and as such is referred to as a Hybrid Delivery System. This consortium is now in the preliminary design phase and by September 2006 will provide the transit agency with a firm lump sum price to construct the line extensions and thus begin actual construction throughout the city.

As a result of METRO's aggressive schedule, almost every engineering firm in the City is either directly or indirectly involved. Additionally, because the selected alignments for the LRT extension involve major thoroughfares, numerous private and public underground facilities will be impacted. To accommodate the LRT improvements, the City of Houston has to relocate numerous critical underground facilities. These relocations include force mains, lift stations, and several large diameter water transmission mains including an extremely vital 66-inch water main. Latest METRO estimates for the City's utility relocations range between $250 million and $350 million.

One of the five corridors that Metro is planning to construct their light rail system along is the Richmond Avenue corridor. The 8.3 miles Richmond Avenue corridor is designed to provide connection from University of Houston Central Campus, Greenway Plaza to the Uptown Galleria area as shown in Figure 2 below.

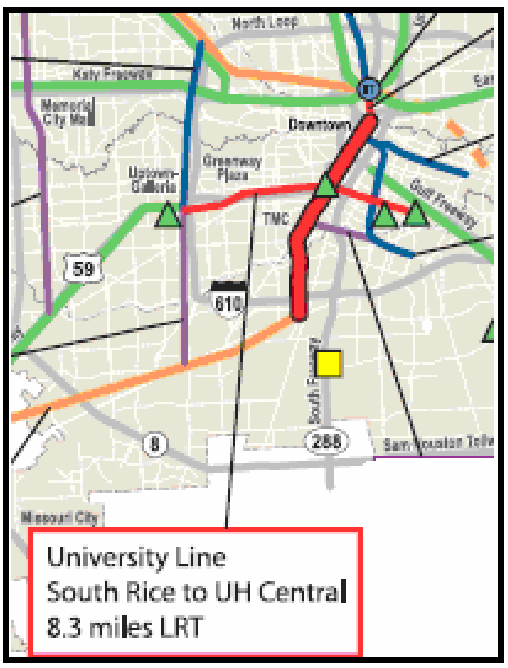

Figure 2. Richmond Line alignment

Criticality of 66-inch Transmission Main

The University LRT alignment will impact the existing 66-inch water transmission line along Richmond from Hutchins to Buffalo Speedway, or approximately 4 out of the 8.3 miles of the

LRT line. The existing 66-inch water transmission main was constructed between 1988 and 1990. This line consists of prestressed concrete cylinder pipe (PCCP) and was installed with an impressed current cathodic protection system. The water main is generally located within the center of Richmond. Other than some isolated leaking joints, this critical line has had relatively no maintenance issues.

The existing 66-inch transmission line is a major source of water supply to the west side of the City and is the sole source of treated water to the City's Southwest Pump Station. The Southwest Pump Station is the largest facility remotely located away from the City's three surface water treatment plants. This facility averages more than 75 MGD and has a peaking pumpage capacity of 120 MGD. On site storage capacity at this facility is 30 MG comprised of 5 ground storage tanks. The Southwest Pump Station service area extends over 80 square miles, including two major employment centers and the Houston Galleria, the largest retail center in the southwest – and the City's primary sales tax generator.

The construction of the proposed light rail system on Richmond will inevitably place the rails over the existing 66-inch water transmission line. By placing the proposed rail above the existing 66-inch water transmission line the City has expressed numerous concerns to METRO. These concerns include the affects the LRT system will have on the operation and maintenance of the transmission line. At a minimum, the City's position is that the rails will prevent access to the line for any necessary repairs, thus the overall longevity of the existing waterline will be compromised. Appurtenances that will be impacted by placing the rails on top of the transmission main include; isolation valves, drain lines, air and vacuum valves and access manways.

Issues with 66-inch Prestressed Pipe

PCCP provided for this 66-inch line was manufactured according to AWWA C301-85, Appendix B. Figure 3 below provides a cross section of this composite material. PCCP is comprised of a steel cylinder embedded into a concrete core and wrapped with prestressing wire and a mortar coating. At the time of its manufacturing, many of the current AWWA standard requirements were not in place such as absorption rates for the mortar. This line was provided with shorting straps and installed with bonded joints.

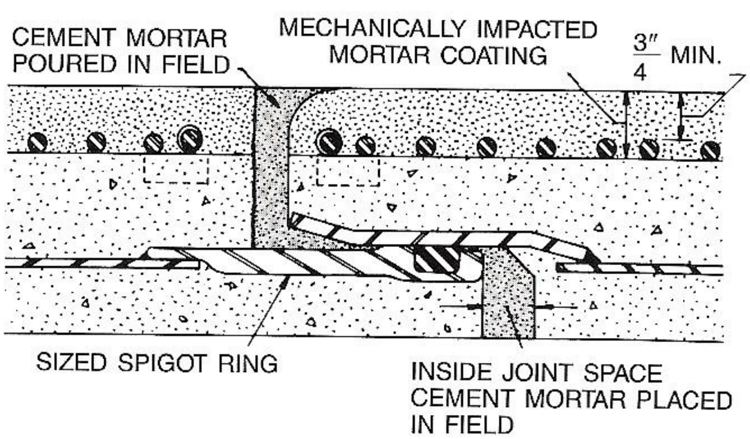

Figure 3. Cross Section view of PCCP

Since the early 1980's the water works industry has experienced some highly publicized failures. Many of the failures have been attributed to hydrogen embrittlement of the prestressing wires. Prestressing wire used in the production of PCCP manufactured in the early to mid 1970's most likely would have been produced in accordance with the standards in ASTM A648-73. While ASTM A648-73 identified Class III wire as an approved tensile class, there was no upper tensile limit for that class in that standard. Upper limits on tensile strength were not incorporated in ASTM A648 until 1984.

In addition, the manufacturing standard did not include torsion test requirements for the prestressing wire. Torsion test requirements did not become part of the standard until 1988. The American Concrete Pressure Pipe Association has collected data on the performance of concrete pressure pipe, which includes pipe with prestressing wire that was manufactured in accordance with ASTM A648-73. Figure 4 below identifies the reported projects with problems as the year that project was either manufactured or installed. As noted in the graph, an increasing trend in the number of projects with problems during the period between 1968 and 1980, in the figure below.

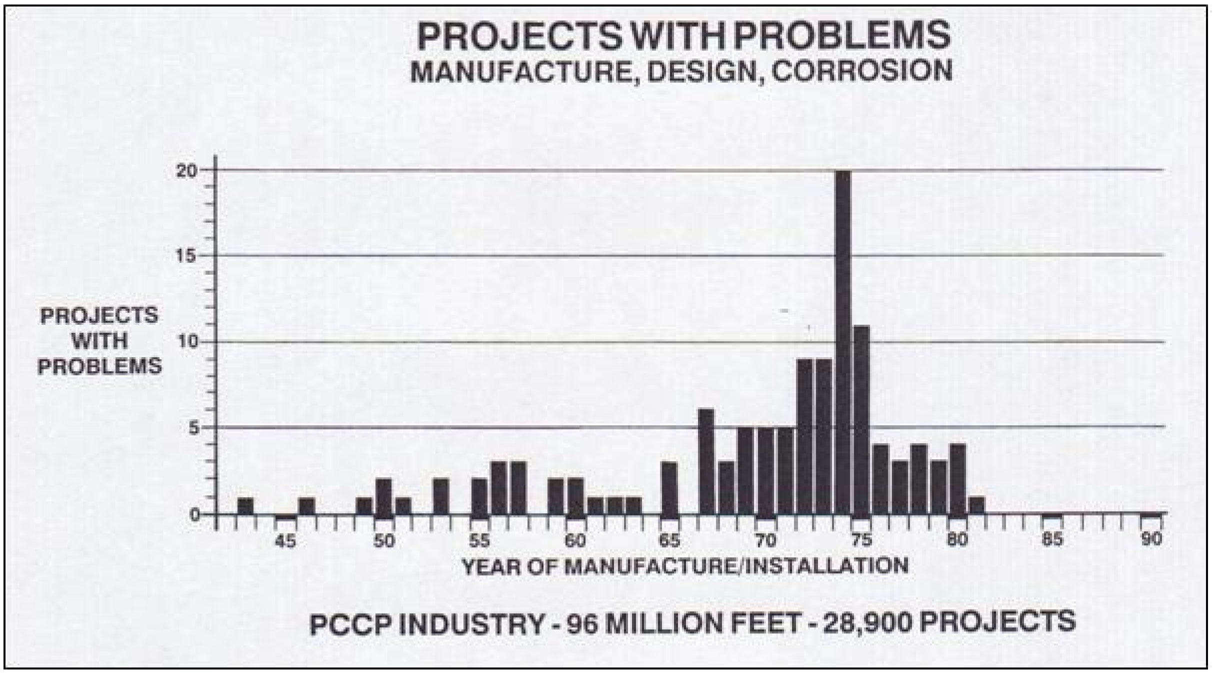

Figure 4. Yearly analysis of PCCP pipe

The investigations conducted on these failures have indicated that the prestressing wire failed at these locations. Tests conducted on the prestressing wire have indicated deterioration from strain aging and hydrogen embrittlement.

The nature of PCCP failures are mostly known for their catastrophic manner. Should this type of a failure occur underneath or in the vicinity of the proposed LRT system, the likelihood of impacting the rail operations is certain to occur.

Potential Stray Currents

METRO's current plans call for the LRT to consist of electrically operated trains. The systems will most likely exist via overhead electrical lines. However, as shown on METRO's existing Main Street LRT, the agency has had trouble from the initial start up of the line to prevent stray currents from migrating into the ground and onto the adjacent utilities. METRO has spent several million dollars to rectify the stray currents. Although the currents have been reduced, to date METRO has not been able to isolate all of the stray currents.

The 66-inch line was installed with cathodic protection in the form of impressed current system. Although this is the most common method of protecting PCCP in low resistivity soils with stray currents, this method has shown not to be totally reliable for the City of Houston. Figure 5 below, shows a section of deteriorated 48-inch PCCP manufactured around the same time as the Richmond 66-inch line. The distress in the 48-inch line was just discovered in February 2006. As noted in the photograph below, Figure 5, there are both longitudinal and circumferential cracks within this 15 year old pipe.

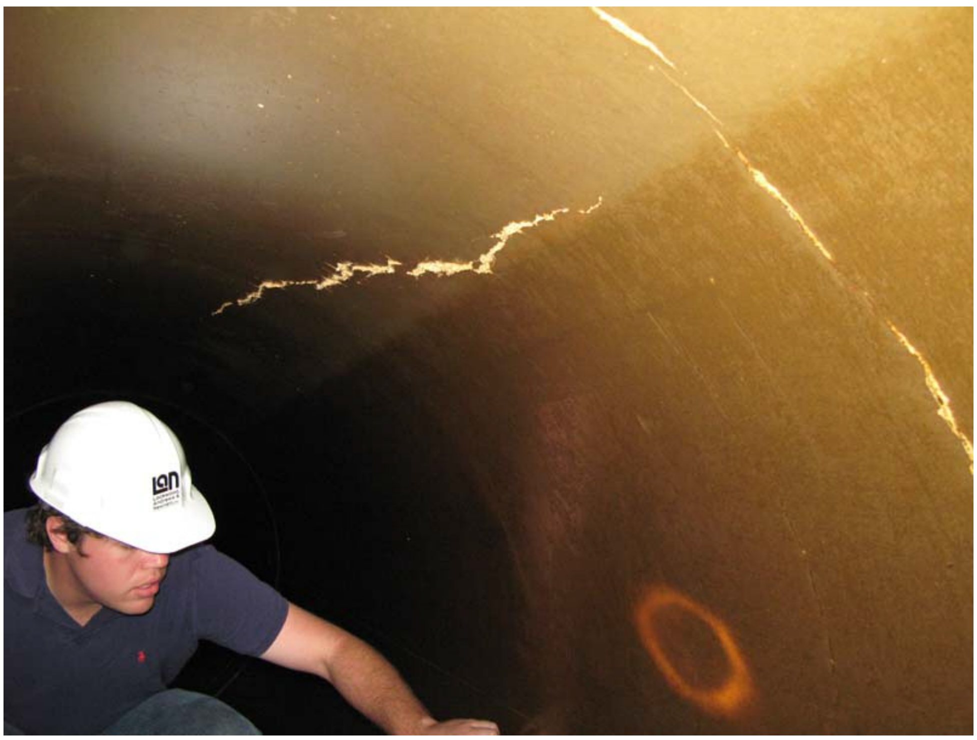

Figure 5. Longitudinal and circumferential damage in PCCP pipe

The proposed light rail line's system's electrical currents will cause problems with the existing 66-inch water transmission line even though it is cathodically protected. Regardless, the deep anode ground beds which accompany the rectifier units will be expended sooner than originally designed.

The $30 Million Question

Currently, the City and METRO are in discussions on how to minimize the potential failures on this line. It is METRO's position that the potential for stray currents is very minimal and the LRT system can incorporate designs that address the City's operational concerns and access issues to the transmission main.

The City is taking the position that even with a reduced potential for stray currents, the criticality of this major transmission main and the mode of failure requires that the 66-inch line to be located outside the same corridor as the LRT system. Complicating matters further is the cost to relocate 4 miles of 66-inch line in a highly urbanized corridor. Preliminary estimates to relocate the 4 miles are shown to be around $30 million. The City of Houston has not provided monies within the Capitol Improvement Budgets for this effort. In addition, the potential alternative alignments for routing the replacement 66-inch line are limited and will require significant portions of the line to extend through residential neighborhoods. The roadways throughout these established neighborhoods are lined with very mature trees that provide a cover over the roadway.

Further Complications

With the five proposed corridors being constructed simultaneously, there is a critical demand for coordination of large diameter City of Houston utility shut downs and outages. In addition to the Richmond 66-inch water transmission main, there are at least another five (5) major water mains that will be impacted. These lines range in size from 36-inch to 84-inches.

Due to subsidence issues, the City was required by a state Subsidence District to convert from primarily ground water, to 80% surface water. Failure to achieve this annual goal results in the Subsidence District assessing a "groundwater pumpage disincentive fee" of $3.00/1,000 gallons. This fee is over twice the City's cost to produce and transport surface water to its customers. When these critical lines are taken out of service, the City is likely to exceed its allowed ground water allocations.

Finally, due to limited CIP funding for the transmission system, the City does not have redundant lines for each of these critical lines. Such shut downs must be limited to the months of November, December and January only. Also, based on past experience, it takes between 30 and 90 days to coordinate a shut down of a critical line. The coordination includes how the City will maintain the minimum required system pressures, satisfactory water quality and chlorine residuals, and necessary notifications to the affected businesses.

Summary

When a train comes, you have to move. The problems are further complicated when the train will impact multiple water transmission mains in several sections of the City. One of the impacted lines is the single most critical line in the City of Houston's system. Although the cost for relocating 4 miles of 66-inch transmission main is estimated at $30 million, this amount does not include the additional cost the City will incur to coordinate the required shut downs and other related cost such as the Subsidence District's disincentive fee. With transit improvements comes the mostly hidden cost to the public utility system.

PROVEN ELECTRICAL TEST METHODS FOR THE EVALUATION OF THE CONDITION OF EXISTING METALLIC PIPELINES

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Abstract: This paper presents a number of case histories described in generic terms without disclosing the specific agencies involved in which cement mortar coated pipelines were damaged and how the extent of the damaged areas was identified. One project was in California, one in Texas and another in Oregon. Typically the mortar coating provides corrosion protection for the steel cylinder; however, once the mortar has become delaminated from the steel cylinder, it no longer provides the same level of corrosion protection.

The close interval survey test method is a non destructive test method that uses electrochemical techniques to identify areas where corrosion activity is present. Areas of corrosion activity are expected to correlate closely with areas of Delamination. The electrochemical technique has the advantage of being able to locate areas of corrosion non-destructively.

The close interval survey method requires the pipeline to be electrically continuous along the length of the survey. However, electrical continuity is not required to complete the evaluation. If a portion of the pipeline is found to be electrically discontinuous, a cell-to-cell survey can be conducted over that portion of the pipeline. A cell-to-cell survey is similar to the close interval survey but does not require electrical contact to the pipeline. If performed correctly, a cell-to-cell survey can provide data which is just as accurate as the close interval survey technique. The paper present data on cement mortar lined and coated steel pipelines however this test methodology can be used on dielectrically coated pipelines also.

1. GENERAL BACKGROUND

The electrical test methods described were used to determine the condition of 12 miles of 24 inch cement mortar lined and coated non continuous water transmission piping in Texas, 5,000 feet of 24 inch cement mortar lined an coated continuous reclaimed water pipeline in California and 20 miles of 42 inch continuous and non continuous cement mortar lined and coated steel water pipelines in Oregon.

2. TEST METHODS

2.1. Electrical Continuity Testing

Electrical continuity testing was performed to determine if the pipe joints of the pipelines are electrically continuous. Electrical continuity of a pipeline is important if cathodic protection is to be applied to the pipeline. For a typical project, pipeline electrical continuity is important because test methods to identify areas of likely corrosion are selected depending on the pipeline electrical continuity characteristics. Traditional CIS test methods may not be effective in identifying areas of active corrosion on electrically discontinuous pipelines.

A temporary impressed current cathodic protection system was applied at available test stations and blow off valves at various locations along the pipelines. A portable DC power source and temporary anode was used to impress a current onto the pipe section under test.

Pipe-to-soil potentials were taken before and during the continuity testing to determine the level of potential shift along the pipeline. Electrical continuity is indicated when the pipe-to-soil potential is made more negative by the application of test current. No potential shift or a shift in a positive direction indicates an electrical discontinuity between the location of the application of test current and the potential measurement location. Pipe-to-soil potentials were taken using a digital voltmeter and a portable copper, copper-sulfate reference electrode.

2.2. Pipe-to Soil Potential Measurements at Existing CP Test Stations

Pipe-to-soil potentials are measured throughout the course of the project field work as test stations are observed in the field. Pipe-to-soil potentials are measured at CP test stations on foreign pipelines. The purpose of these measurements on foreign pipelines was to determine the level of cathodic protection applied on the pipelines in the vicinity of the measurement location. This data is used to determine the likelihood of stray current interference from foreign cathodic protection systems onto the pipeline being evaluated.

2.3. Cell-to-Cell Close Interval Survey

Cell-to-cell close interval surveys (CCCIS) were conducted on all of the pipeline segments that were determined to be electrically discontinuous. The purpose of the CCCIS is to locate areas of likely corrosion along the surveyed pipeline.

Cell-to-cell close interval surveys are conducted by measuring a potential gradient in the earth using two copper-copper sulfate reference electrodes and a high-impedance digital voltmeter. The measurements are stored electronically in a field computer along with the estimated pipeline station. GPS coordinates are collected at various intervals depending on project conditions and, depending on satellite availability.

Copper, copper-sulfate reference electrodes consist of a plastic tube with a porous ceramic plug. The plastic tube contains a copper rod and a copper sulfate solution. The reference electrodes are placed approximately 5 feet apart along the center line of the pipe. When access to the center line of the pipe was not possible, the reference cells were placed at an off set to the centerline.

As measurements are taken, the reference cells are repositioned every 5 feet along the pipe. The reference cell over the pipe centerline ahead of the survey crew is connected to the positive voltmeter terminal and the reference cell closest to the survey crew is connected to the negative terminal. In this manner, positive polarity measurements indicate areas where current is flowing off the pipe surface and into the electrolyte (anodic or corroding location). Negative polarity measurements indicate current flow onto the pipeline (cathodic or non-corroding location).

A typical graph of a cell-to-cell survey showing an anodic area on a pipeline is shown in Figure 1. As can be seen on the graph, as the survey data is collected from left to right, a number of negative readings (cathodic or non-corroding) are recorded. As the area of active corrosion is approached, the cell-to-cell potentials become more negative then quickly change to positive (anodic or corroding) with a positive "spike" occurring at the area of active corrosion. After passing the corroding area, the potentials return to more negative values.

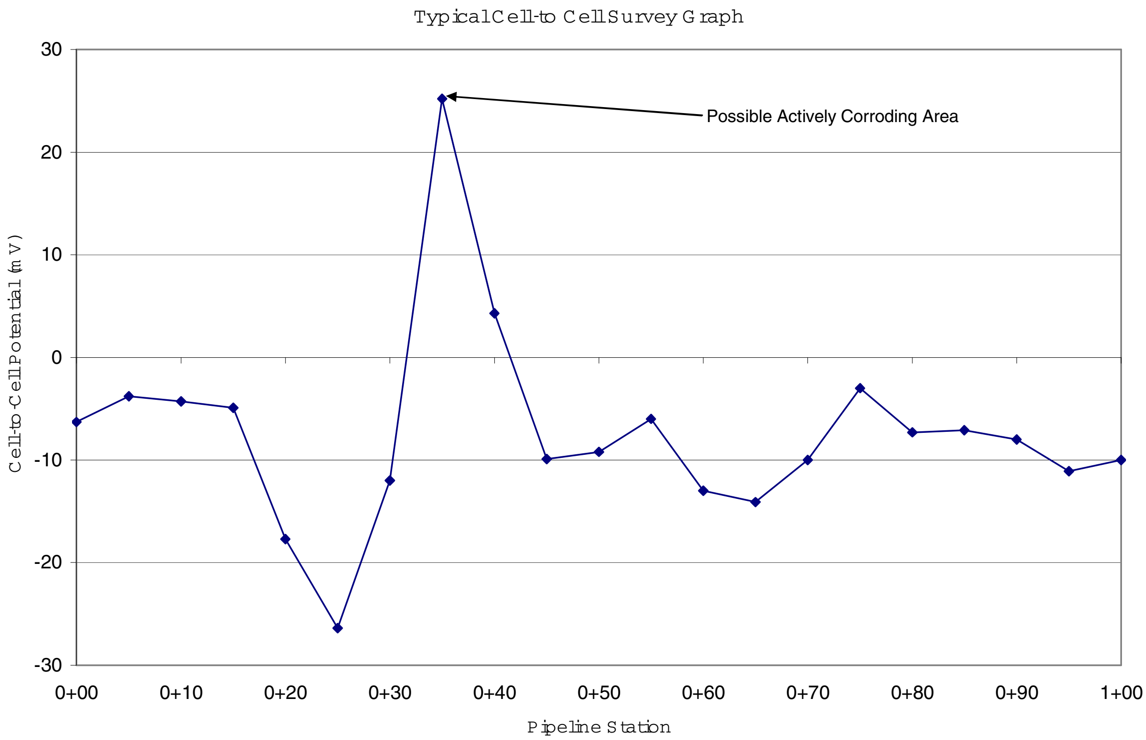


Figure 1

2.4. Typical Pipe-to-Soil Potential Close Interval Survey

The PSCIS on an electrically continuous pipeline is used to identify areas along the pipeline that show a significant negative potential shift over a short distance. These areas typically indicate areas of current flow off the pipeline and into the electrolyte (corrosion).

On pipelines that are not electrically continuous, the PSCIS survey is essentially the same as a CCCIS. Instead of using two portable copper, copper-sulfate reference electrodes, the section of pipe connected to the voltmeter acts as one of the reference electrodes. It acts as a reference electrode because it has an essentially stable value during the course of the survey. The result is a fixed reference cell to a moving reference cell PSCIS. The data analysis is similar to the CCCIS. However, the magnitude of the data for the PSCIS is in the 0.1 to 0.6 volt range and in the 0.001 to 0.1 volt range for the CCCIS data.

The PSCIS uses the same equipment as a cell-to-cell survey with the exception that only one portable copper, copper-sulfate reference electrode is used and a continuous connection to the pipeline is made at an available appurtenance. Data evaluation for the

PSCIS survey on continuous pipelines is different than on discontinuous pipelines. On continuous pipelines not under cathodic protection, anodic (corroding) areas are indicated by large negative potential shifts in the PSCIS over a short distance. On discontinuous pipelines, anodic (corroding) areas are indicated by positive potential shifts over a short distance.

2.5. Side Drain Measurements

During the CCCIS and PSCIS surveys, side drain measurements are conducted at locations where indications of possible corrosion activity are initially indicated. The purpose of the side drain measurements is to quantify the magnitude and direction of the current flow on each side of the pipeline.

Side Drain measurements are taken by placing the positive reference cell over the centerline of the pipe and the negative reference cell 5 feet off to the side or perpendicular to the centerline of the pipe. In this manner, positive polarity side drains indicate areas where current is flowing off the pipe surface and into the electrolyte (anodic or corroding location) and negative polarity side drains indicate current flow onto the pipeline (cathodic or non-corroding location).

2.6. Soil Resistivity Measurements

Understanding how easily current will travel through a medium surrounding a metallic object is important in evaluating the corrosive environment. Soil resistivity measurements provide this information by quantifying the electrical resistivity of the soil to a certain depth. Soils with lower resistivity are considered more corrosive than higher resistivity soil.

Resistivity is an inverse measure of the ability of a soil to conduct an electric current, with higher resistivity resulting in a lesser degree of current flow. Corrosion rate depends on current flow between a metal and the adjacent medium. Normally, the corrosion activity on metals in soil increases as soil resistivity decreases. The following Table 2 correlates resistivity values with degree of corrosivity. The interpretation of corrosivity correlation to soil resistivity varies somewhat among corrosion engineers. However, this table is a generally accepted guide.

Table 2 Soil Corrosivity[1]

Soil Resistivity (ohm-cm)	Degree of Corrosivity
< 500	Very High
500 – 1,000	High
1,000 – 2,000	Moderate
2,000 – 10,000	Mild
> 10,000	Negligible

3. CONDITION VERIFICATION LOCATIONS

Survey data, soil resistivity data, and consideration of the area where the survey was conducted was used to select locations for condition verification inspection. CIS techniques were performed along all of the pipeline segments. CCCIS data was analyzed to determine possible areas for condition verification inspections. Further CCCIS were performed at the locations selected for possible condition verification to confirm the likelihood of corrosion at these locations and to confirm the accuracy of the initial data collected.

From the data collected and reviewed locations are selected as candidates for condition verification. It is noted that the survey techniques used during the survey may find corrosion on other nearby structures rather than the targeted pipeline. It is also noted that no corrosion may be present at any of the identified pipeline locations.

While efforts have been made to exclude locations that initially indicated possible corrosion activity due to extraneous factors, it is not realistic to know positively that corrosion is occurring at a given location. The attempt of this project is to identify, through the use of proven electrical survey methods, the areas most likely to exhibit corrosion. Proof of the presence of corrosion at any of these locations can only be verified through excavation and inspection.

There are a number of factors that can provide false indications of the presence of corrosion. Some of these factors are as follows:

[1] Peabody, A.W. and Parker, M.E., "Corrosion Basics, an Introduction," Ed. by Brasunas, A. de S., NACE International, p. 191 (1984)

- CCCIS Data not collected over the centerline of the pipe which can lead to measuring earth current flows from other structures

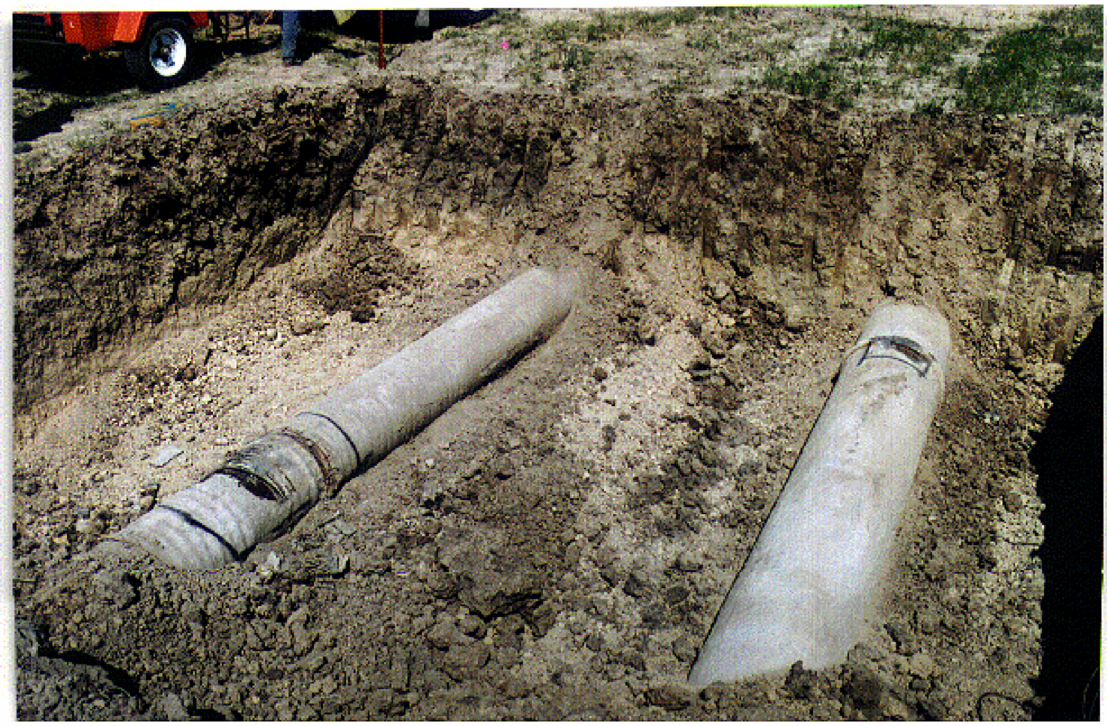
- Stray electrical earth currents present in a survey area. CCCIS data would measure the earth current flow but not identify source.

- Unknown underground metallic structures present in survey areas exhibiting positive potential shifts in CCCIS.

- Unknown cathodic protection systems or galvanic anodes installed on foreign pipelines in the survey area.

- Poor earth contact of the copper-copper sulfate reference electrode to the soil gives potential measurements more negative than actual.

As an example of how an underground metallic structure can influence the survey data, large metallic appurtenances such as manhole covers that are approximately 3-foot diameter made of cast iron could influence the readings from these test methods.



Each location is ranked according to the likelihood of finding significant corrosion. The risk ranking was based on:

1) Soil corrosivity from the soil resistivity data collected by other

2) CCCIS data exhibiting typical anodic indications

3) The level of confidence that the survey data was representative of the targeted pipeline and not of some other foreign structure.

The total risk ranking is a sum of these factors. A numeric value is assigned to ranges within these three areas. The higher the total point value, the greater the likelihood that corrosion is occurring at a given location. The numeric values assigned to each factor are shown on Tables 4, 5, and 6 below:

Table 4, Soil Resistivity Risk Ranking

Soil Resistivity	Point Value
10,000 -15,000 Ohm-Cm	1
6,001 – 9,999 Ohm-Cm	2
3,000 – 6,000 Ohm-Cm	3
1,000 – 2,999 Ohm-Cm	4
0-999 Ohm-Cm	5

Table 5, CCCIS Risk Ranking

CCCIS Indication	Point Value
1-10 mV	1
11-20 mV	2
21-30 mV	3
31-40 mV	4
41 mV and Up	5

Table 6, Survey Area Risk Factor

Survey Area Factor	Point Value
> 10 feet off pipe center line, other utilities present	1
1 to 10 feet off pipe center line, other utilities present	2
1 to 10 feet off pipe center line, no other utilities present	3
Over pipe center line, other utilities present	4
Over pipe center line, no other utilities present	5

The locations with the highest risk of corrosion, the ranking for each location by risk factor and the sum total for each location are shown below in Table 7 for a hypothetical pipeline 1.

Table 7
Risk Ranking

Location	Pipeline	Station	Soil Resistivity Rank	CCCIS Indication Rank	Survey Area Factor	Total Risk (max 15)
1	1	10+00	4	5	4	13 (93%)
2	1	28+29	4	3	5	12 (80%)
3	1	39+09	4	2	5	11 (73%)
4	1	42+49	3	3	4	10 (66%)
5	1	51+29	4	2	4	10 (66%)
6	1	67+06	3	1	5	9 (60%)
7	1	69+30	4	2	3	9 (60%)
8	1	71+01	4	3	2	9 (60%)
9	1	75+81	4	3	1	8 (53%)
10	1	76+56	4	2	2	8 (53%)
11	1	88+71	4	2	2	8 (53%)
12	1	91+82	4	2	1	7 (46%)
13	1	93+27	4	2	1	7 (46%)
14	1	94+66	4	2	1	7 (46%)
15	1	94+05	4	1	1	6 (40%)

4. FINDINGS AND CONCLUSIONS

The result of these test methods resulted in the determination that the majority of the pipeline alignments were in generally good condition and the damage to the cement mortar coating was site specific and not systemic.

Generally if the soil is not significantly aggressive, the damage is generally limited to areas where the cement mortar is damaged during installation or by third party damage. Another finding of these test methods is that if the cement mortar is not installed at fitting and specials it will result in corrosion of the exposed steel in a very short period of time.

The electrical test methods presented give the pipeline operator a very cost effective way to determine the condition of a buried asset with minimal impact to the pipelines and the surrounding area along the route alignment.

Combining the electrical test methods with the risk analysis presented, the location for the excavation of the pipeline to document the actual in the field condition can be optimized. By this is mean that the location of the selected excavation will have the greatest likelihood of having some corrosion. Once the pipeline is exposed the actual condition can be documented and the appropriate repair actions can now be taken by the pipeline owner.

Analysis on the Advantage of Trenchless Construction's Cost by Disutility-Cost Assessment Method in China

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Abstract

Trenchless technology (TT) methods, including horizontal directional drilling, pipe jacking and pipe renewal are advanced methods for installing, repairing and renovating of underground pipes, ducts, cables, etc., with minimum excavation of ground surface. In China, the number of small-scale projects using horizontal directional drilling (HDD) equipment has increased quickly, while the number of large-scale project by pipe-jacking is increased slowly, and there are only a few projects using pipe replacement (renewal). One of the important obstacles in growth of trenchless technology is the difficulty in estimating the cost of trenchless construction, including the cost to general public, environmental impacts, damage to pavement and underground existing utilities and structures, when compared to the open-cut methods. This paper puts forward a disutility-cost assessing method combined with linear regression method. Firstly, the total disutility of trenchless construction can objectively be estimated by quantitative analysis on the factors related to trenchless construction. Then, on the basis of the assessment we derive a model to calculate the cost and obtain the characteristic parameters and empirical constant of the cost model by means of linear regression method. Finally, the cost advantage of trenchless construction will be worked out by comparing with other construction methods of underground utilities. This paper is helpful for engineers to make a rational decision among underground utility construction methods.

Keywords

Trenchless technology, disutility-cost, linear regression method, cost model

Introduction

Developing rapidly since the first international No-dig conference in 1996, Trenchless technology has been widely used in most Chinese cities with the continuously increasing market share of pipe installation and maintenance. Figure 1 shows that the total pipeline project output values used with trenchless technology increased sharply from 1.2 billion RMB in 2000 to 6.79 billion RMB ($850 million) in 2005. However, the development of TT is unbalanced in China. For example, TT for pipe installation and maintenance is much more popular in southeast areas than west areas. There are much more small-scale TT projects and equipment than the large-scale TT projects and equipment. Unfortunately, there is still a large number of large-diameter water-supply and sewerage lines are installed by open cut method.

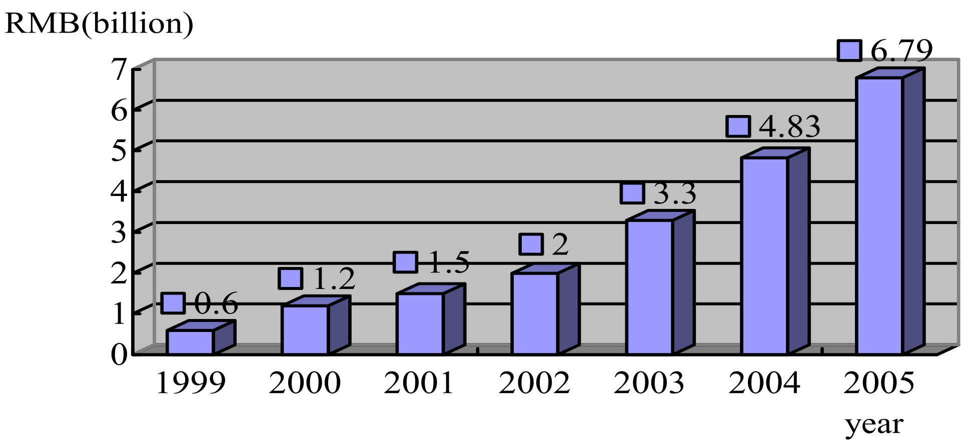

Figure 1. The total quantities value for the trenchless technology industry
($1 = 7.8 RMB)

The analysis on the statistical data of trenchless unites leads to the tendency of the development and output values of TT industry in China. As showed in Figure 2, there are 2546 Horizontal Directional Drilling (HDD) machines including 1574 small-scale HDD rigs (less than 20 tons), 45 medium-scale HDD rigs (between 22 and 43 tons) and 61 large-scale HDD rigs (more than 45 tons) as well as 370 pipe jacking machines in 2005 in China. The number of small-scale HDD rigs is extraordinarily larger than the others. Besides, the growth rate of small-scale HDD rigs from 2001 to 2005 is also rapid while the others are slow and smooth. The primary application of trenchless technology, in China, is the small-diameter pipe installation. Contrarily, the application of trenchless technology for large-diameter pipe is limited because of the relatively high cost, and its market share is not only smaller than trenchless installation for small-diameter pipe, but also pretty less than that in the developed countries such as North America, Germany. What is more, 87% of the market share of large-diameter pipe installation by trenchless methods is river or road crossing, which cannot be constructed by the open cut method, and just 9% of the market share is to avoid interrupting traffic and destroying the underground utilities.

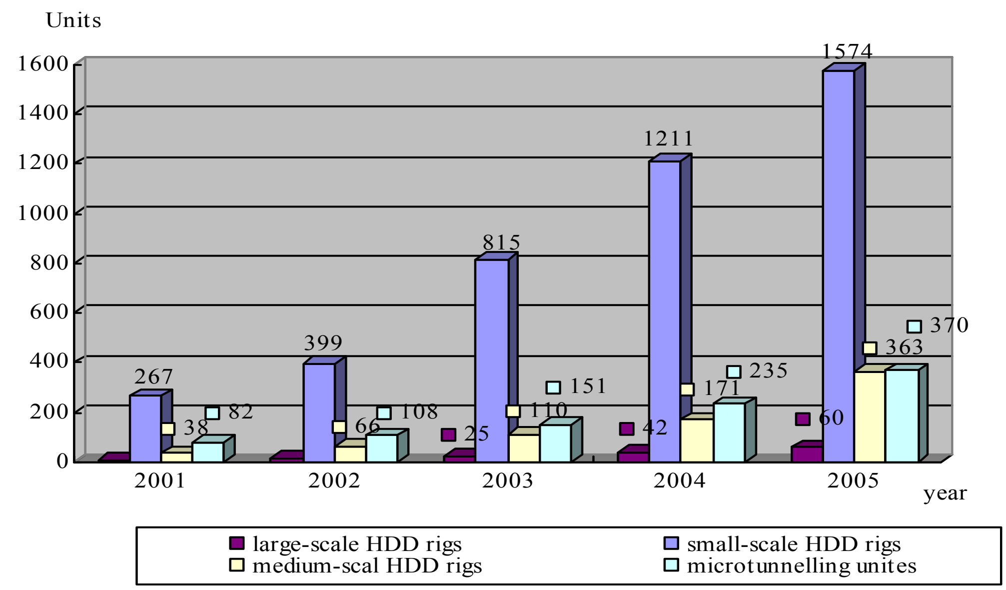

Figure 2. The number of different trenchless units from 2001 to 2005

Underground construction industry has realized why it is difficult to get more market share of large-diameter pipe installation by trenchless. Some of the viewpoints are justly based on the technology problem, such as the lack of monitoring and steering technology, power system and cut-away system. Others think that the machines used in large-diameter pipe installation, pipe jacking and pipe replacement is too expensive to be invested by some enterprises. Investigation by the CSTT (China Society for Trenchless Technology) shows that the development of TT is fast from 1998 to 2006, and has achieved the level of the developed countries. There are many enterprises with good finance in China such as Shanghai Tunnel Engineering Corporation (STEC) and so on.

In summary, the lack of related technology and capital are not the real reasons that hinder the application of trenchless technology for large-diameter pipe. So what are the key problems for it?

Cost Analysis on TT in China

Unreasonable assessment on the cost of pipe installation, especially in the urban areas with heavy traffic and large population, results in the slow development of TT for large-diameter pipe in China. The cost analysis on the different scales of TT project will approve this conclusion. It is important to note that the 'cost' of pipe installation, in China, just includes the expenditure of pipes and materials as well as labor costs, but does not include the compensation on the negative effect on environment and traffic. There is still no requirement for the compensation on the negative impact on the urban areas and their residents, so it is the reason that open cut method is still the most popular method for pipe installation in China.

As shows in Figure 3, the cost curve of small-diameter pipe installation has dropped down very fast and nearly come to the rock bottom that is the 1/4 of that in 2000 until 2003. For example, when the pipe diameter is not larger than 200 mm (8 in.) and the installation depth is 2 m (6.5 ft), the cost for pipe installation by TT is lower than open cut method in 2005. By comparing Figure 2 and Figure 3, it is easy to find that when the cost for small-diameter pipe installation by TT declined to the rock bottom and is nearly equal to the cost by open cut method in 2003 in China, the number of small-scale HDD rigs increased rapidly from 399 in 2002 to 815 in 2003.

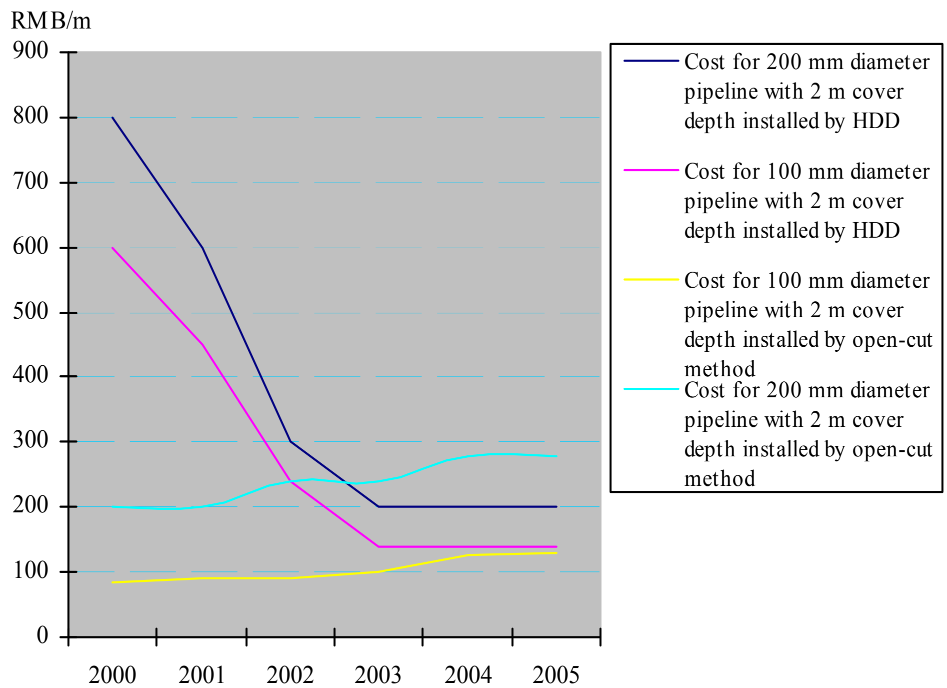

Figure 3. The cursory average cost of small-scale pipe installation in China ($1 = 7.8 RMB)

At the beginning, the advantages of TT attracts the investment on trenchless machines for small-diameter pipe installation which is very cheap and gets the support of Chinese government, then with the gradual perfection of constructional technology and automated machines, the cost for pipe installation by TT reduces until up to the point equal to open cut method. Therefore, the trenchless technology takes up the vast market share in small-diameter pipe installation. So it not only has the advantages for traffic and environment of the urban areas, but also for the cost which contributes to the development of TT for small-diameter pipe installation.

Unfortunately, the development of trenchless technology for large-diameter pipe installation is held back because of the high cost, compared with open cut method. There are several reasons. First of all, the cheap and superfluous labor force in China reduces the cost of open cut method for large-diameter pipe installation. As shown in Figure 4, the cost between the TT and open cut method for large-diameter pipe installation is quite different, nearly 5 times. Second, the large capacity drilling machines, which usually owned by some state-owned companies, are very expensive (about 1 million RMB for a 25-ton HDD rig), this is unfavorable for the development of the trenchless market. Last but the most important factor is that it is difficult for the government to quantify the compensation for the traffic interruption and environmental damage caused by open cut construction and so far there are no laws and regulations for that. It is not correct to determine which method is the best one for a pipe installation or maintenance project in an urban justly by comparing the direct cost such as the expenditure of materials and the cost of labors, because the cost for the effect on the environment and traffic in the urban is also considerable. The total cost for pipe installation must consist of both the former and the latter.

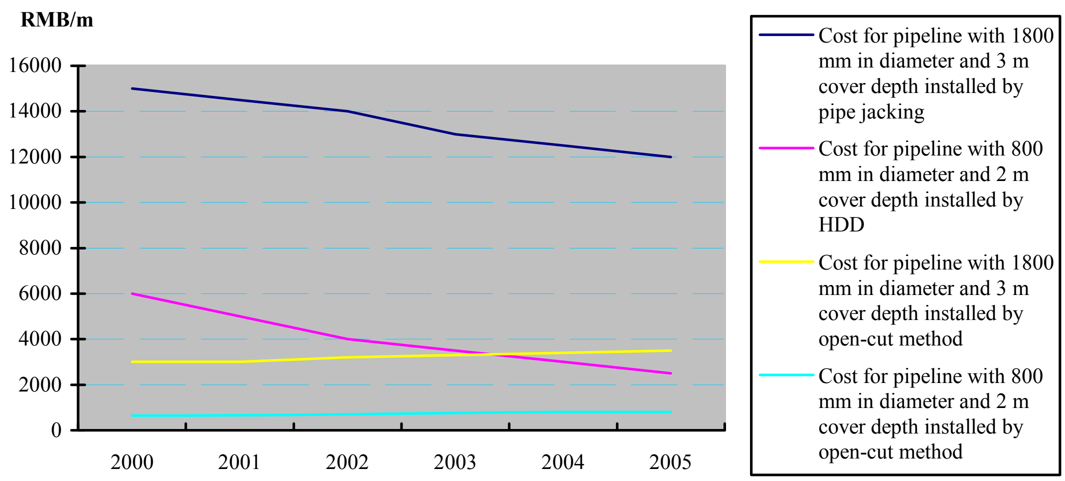

Figure 4. The cursory average cost of large-scale pipe installation from 2000 to 2005 in China ($1 = 7.8 RMB)

So the key to encourage the application of TT for large-diameter pipe installation is to quantify the effect by pipe installation on the traffic and environment in Chinese urban areas. By doing this, the government will have some standards to decide which way is the best one for constructing a pipeline. In this paper, a feasible way to quantify the effect on the negative factors caused by the installation of underground pipe will be introduced. If the accurate evaluation on which is the best means for a large-diameter installation project between trenchless and open cutting method given to the government, it would be in favor of the development of TT.

The Disutility-Cost Assessment Method

In order to give a reasonable economic evaluation on the underground pipe installation in a city, this paper will introduce and use a microeconomic utility. Based on this theory, the formal concepts of "utility" and "disutility," as well as those of "benefit" and "effect," will be introduced. Activity, such as instruction of freeway and installation of pipeline, will lead to either positive effect or negative effect, and positive effect can be regarded as "utility" while negative effect regarded as "disutility." Theoretically, the cost of an activity must be equal to that of the disutility, but it is impossible to anticipate all disutility of an activity. Therefore the cost of an activity, including social and natural cost, can be regarded as the subjective reorganization to the disutility. An all-round analysis on both the utility and disutility of a project activity will lead to correct assessment and decision on the working practice. In the field of TT, there are many criteria and methods to analyze the disutility of pipe installation in developed countries according to the situation of these countries, so it is easy to decide which technology for pipe installation has more economic benefits. However, because of the lack of the evaluation method and statistical data about pipe installation in urban areas, it is difficult for the government of China to set exact detailed criteria. For example, in Wuhan, the local government does not allow the open cut method for pipe installation in the crowded streets that almost could not be interrupted by any construction, rather than decide which method for pipe installation and maintenance according to the all-around evaluation on the economic benefits. Consequently, the creation of an evaluation method for pipe installation will promote the development of TT in China.

Considering the activity of pipeline installation, the utility is the aim of this project that the pipeline will perform its function to transmit liquid or gas needed by residents in the city. While the disutility consists of two main parts, one is cost for construction such as material consumption and labor force, and the other is the negative effect on the society and nature. All the disutility can be considered and investigated in the following list.

- Material for pipe and other accessorial appliance as well as the employment of workers
- Direct effect on the environment, such as road pavement, roadbed, and nearby structures.
- Indirect effect on the environment, such as dust, vibration, and noise.
- Effect on traffic in urban areas.

Based on the analysis, the total disutility function for pipe installation in urban areas can be formulated by Formula (1):

$$D_{(pipeline,m),\hat{K}} = \sum_{k=1}^{k} (C_{(construction,m),k} + E_{(environment,t),k} + T_{(Traffic,t),k}) \qquad (1)$$

Where: $D_{(pipeline,m),\hat{K}}$ = The total disutility for a pipe installation, renewal and replacement project that consists of k different segments; $C_{(construction,m),k}$ = Pipe material and other accessorial appliance as well as the employment of workers for the k segment of the project; $E_{(environment,t),k}$ = Direct and indirect effect on the environment during the k segment construction of the project; $T_{(Traffic,t),k}$ = Effect on the traffic during the construction on the area where k segment of the project is located; m = The means, by which pipelines are installed, renewed or replaced in line; t = The time, when the k segment of project is completed.

The relationship between the cost and the disutility is ascertainable. Project cost can be regarded as the subjective recognition to the disutility, which is invariable. What is more, the degree that the each part of the formulation is subjective recognition is different. For example, the part $C_{(construction,m),k}$ of $D_{(pipeline,m),\hat{K}}$ is usually consentaneously recognized and easy to be evaluated while it is difficult to come to an agreement to recognize both $E_{(environment,t),k}$ and $T_{(Traffic,t),k}$ which are often ignored in the cost evaluation of the project in China. So the relationship between the cost and the disutility can be easily formulated by Formula (2):

$$Cost^{a}_{(pipeline,m),\hat{K}} = \sum_{k=1}^{k} (a^{a}_{C,k} C_{(construction,m),k} + a^{a}_{E,k} E_{(environment,t),k} + a^{a}_{T,k} T_{(traffic,t),k}) \qquad (2)$$

Where: $Cost^{a}_{(pipeline,m),\hat{K}}$ = Project cost that evaluated by factor a; $a^{a}_{C,k}$ = Coefficient for subjective weighting of disutility on the material and labor force by factor a; $a^{a}_{E,k}$ = Coefficient for subjective weighting of disutility on the environment by factor a; $a^{a}_{T,k}$ = Coefficient for subjective weighting of disutility on the traffic by factor a.

TT has more advantages on the urban environment and traffic than open cut method for pipe installation and replacement, especially in the urban areas with large population in China. In order to have a reasonable evaluation on the project cost, it is important to put forward the detailed estimation method for the disutility $E_{(environment,t),k}$ and $T_{(Traffic,t),k}$.

The Analysis on the Disutility of Environment

Impact on the environment has become one of the most important factors that weight the feasibility of a project. The disutility that is brought by pipe installation and replacement can be divided into four parts: damage to the road surface and roadbed, noise, damage to the nearby structures and other underground pipes, and air pollution. There are several perfect criteria on the compensation for damage to the road and other underground pipes in every city, and the disutility can be evaluated by referring to these criteria. The pollution arising from pipe installation is a continuous process and has a direct relation with construction project duration, so the disutility of environment can be formulated by Formula (3):

$$E_{(environment,t),k} = \iint f_{E,k}(t,l,d)dtdl \tag{3}$$

Where: $f_{E,k}(t,l,d)$ is a disutility-function of construction time *t,* construction length *l,* and construction depth *d.*

Disutility will usually reflect the neglected cost, both for the pollution and damage; on the other hand, the cost-function will also reflect the disutility partly. The relation between disutility-function and cost-function can be formulated by Formula (4):

$$fc_{E,k}(t,l,d) = a^{a}_{E,k} f_e(t,l,d) \tag{4}$$

Where: $fc_{E,k}(t,l,d)$: is a cost-function of *t, l* and *d*

Accordingly, for the purpose of analysis and optimization, cost-function can be used to get an insight in the real disutility related to pipe installation. By regression analysis on the statistical data about the cost for the pollution and damage, the disutility-function can be developed.

The Analysis on the Traffic Disutility

According to the investigation about the pipe installation in China by CSTT, the impact on the urban traffic is the most serious problem in the Chinese urban areas. However, few methods and criteria can be found and used to evaluate the cost and economic benefits of pipe installation. The impact on the urban traffic, caused by pipe construction is continuous and immediate. Factors that have the great contribution to the traffic disutility of pipe construction, suggested by CSTT, include surface area under construction, traffic flow, traffic capacity, and construction location in relation to the road. The construction location can be divided into two different crossing ways between the road and the pipeline: parallel and vertical. The former has primary impact on the passersby; while the latter has primary impact on the traffic and is more serious than the former. The floor area under construction per kilometer (per mile) can be used to weigh the traffic disutility in relation to the construction scale on the road surface. When the parameter *S* in Figure 5 comes to a starting value, the traffic disutility begins to vary and increase with the increase of *S* by index movement. While the parameter *S* reaches to the critical value, the traffic disutility comes to the maximum value. Figure 5 shows the rate of changing curve.

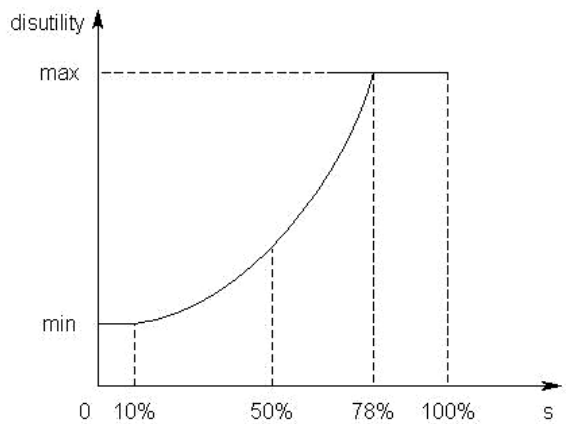

Figure 5. The relationship between disutility of pipe installation and the floor area under construction per kilometer *S*

Ratio (remark *f*) of the traffic flow to the traffic capacity can be used to denote the level of traffic congestion. For a certain pipe construction, the traffic disutility will increase quickly along with the increase of parameter *f*. Figure 6 shows the relationship between parameter *f* and traffic disutility.

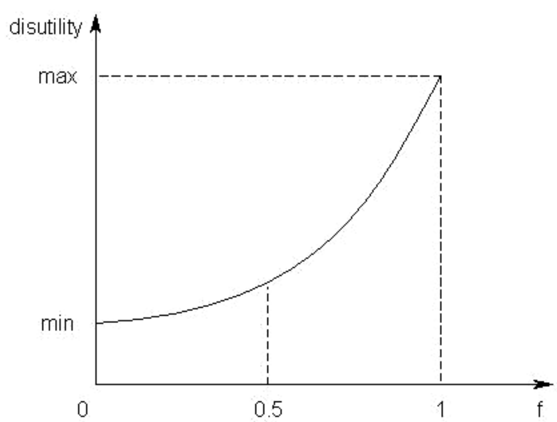

Figure 6. The relationship between disutility of pipe installation and the ratio *f* of the traffic flow to the traffic capacity

According to the analysis on the factor that contributes to the traffic disutility of pipe construction, the traffic disutility can be formulated by an integral representation about time T.

$$T_{(traffic,t),k} = \int f_{T,k}(q, f, s, \eta)dt \tag{5}$$

Where: $f_{T,k}(q,f,s,\eta)$ is a disutility-function of q, f, S, and η; q is the crossing direction between pipeline and road. There are generally two ways of pipe construction: parallel to road and particular. f is ratio of the traffic flow to the traffic capacity, the number of the auto or passersby passing through the road per hour. S is the floor area under construction per kilometer, which is occupied by machines, pipes, workers, and so on, and lost the function of usage. The parameter η is the economic loss coefficient per hour. This coefficient is correlation with the passersby and traffic vehicle drivers. By regression analysis on the statistic data about the cost for the effect on the traffic, the disutility-function can be educed.

Conclusions

According to the analysis of the statistical data about the development of trenchless technology, this paper pointed out the lack of evaluation on the cost of pipe construction results in the unbalanced development of TT in China. A disutility-cost assessment method has been put forward, by which an objective and all-round economic evaluation on the pipe construction suitable can be achieved. A simple formulation about the disutility and cost was developed. The related parameters of the formulation are listed and discussed in detail and the degree of correlation to the disutility of pipe installation is analyzed as well.

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Bibliography

Binsbergen, A.V, Visser J. (2001). Innovation Steps towards Efficient Goods Distribution Systems for Urban Area, *DUP Science*, The Netherlands, 97-103 & 265-272.

He, Y.Z. (1999). “The State and Prospect of Trenchless Technology.” *Symposium on Trenchless Technology Beijing’ 99*, 365(2~3), 3-10.

He, Y.Z. (2003). “2002 Trenchless Technology Market in China.” *Trenchless Technology* (in Chinese), 36(2~3), 1-6.

He, Y.Z. (2004). “China Trenchless Industry Market in 2003—Enormous Growth.” *Trenchless Technology* (in Chinese), 48(2~3), 2-8.

He, Y.Z. (2005). “Trenchless Market 2004 in China.” *Trenchless Technology* (in Chinese), 60 (2~3), 1-5.

He, Y.Z. (2006). “Annual China Trenchless Industry Survey.” *Trenchless Technology* (in Chinese), 72(2~3), 1-6.

Ma, B.S. (2006). “A New Subject of Underground Engineering: Trenchless Technology.” *Trenchless Technology* (in Chinese), 72(2~3), 12-16.

Najafi, M. and Gokhale, S. (2005). Trenchless Technology Pipeline Design, Construction and Renewal, McGraw-Hill, New York.

Stein, D. (2001), Rehabilitation and Maintenance of Drains and Sewers, *Ernst & Sohn Berlin*, 649-654.

Research on Construction Technology for Yangtze-River Crossing Tunnel Project in Wuhan

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Abstract

Yangtze-River crossing tunnel in Wuhan, a key project in China currently under construction, is a twin bore, two-directional, and four lanes tunnel, with outer diameter of 11.38 m (37 ft) crossing the Yangtze-river, the largest river in China. This project uses two Compound Slurry Shields with large section to cross the Yangtze-river. The total length of tunnel is 5,049 m (16,566 ft) so it has long continuous shielding distance. The site geological conditions and the surface environment is complex, because the soil conditions of river bed are consisting of sand stratum, gravel, and shale rock. Around the tunnel job site, there are different kinds of underground pipelines, which makes the tunnel construction more complicated. Other challenges are the requirements for surface settlement and soil deformation control and environmental protection which are very strict. So with the long drive, shallow depth, and high hydrostatic pressure, this project will be facing with many unexpected difficulties. This paper mainly discusses such parts as general features of the project, its major characteristics, and the construction scheme and safety measures.

Keywords

Yangtze-river, tunneling, trenchless technology, underground construction

Introduction

Wuhan Yangtze-river crossing tunnel, the important channel connecting Hankou and Wuchang District, is located between two main bridges. The tunnel begins from the crosslink between the Dazhi Road and Mingxin Street of Hankou District in the northern Yangtze-river, and ends at the Shahu Road planned in the south side of Friendship Street of Wuchang District.

The Yangtze-river crossing tunnel is located on a flourishing area segment, which is densely covered with buildings and underground pipelines. So in order to satisfy the construction requirements, thousands of residents should be moved in advance and a large amount of money has to pay for them. Although the construction area is relatively crowded, but water, electricity, access roads and communication facilities are provided near project site. The geographical location of the tunnel is shown as Figure 1.

Figure 1. The geographical location of Wuhan Yangtze-river crossing tunnel

Project Geology

About the northeast 30 degree, Yangtze-river flows across the area, and the width of the river is about 1,100 m (3,609 ft). The river way is smoother and more unbend, and its main sea route is near the river center. The highest flood water level has reached at 29.7 m (98 ft) in the history. According to the detailed geological investigation data and design documents, the formations would be encountered during construction are mainly clay, silty sand, silty clay, muddy silty clay, silty fine sand, medium-coarse sand, boulder bed, argillaceous silty sandstone with sandstone and shale layers.

The surface water systemm in this area mainly includes Yangtze-river water system, Han jiang river system and Shahu water system. The Yangtze-river water system is the main water system in this area, and is supposed to have important influence on the tunnel construction. While the underground water mainly has three types: upstream water, pore water and berock crevice water. The anti-earthquake degree of the tunnel is designed to be 6 degrees.

The Major Characteristics of the Project

The establishment of this tunnel is to meet the demand of rapid development of city transportation, the demand of supporting the balanced development of both sides of Yangtze-river, the demand of improving the investment environment and promoting the rapid development of the economy of the whole city. So this project is very important to Wuhan city

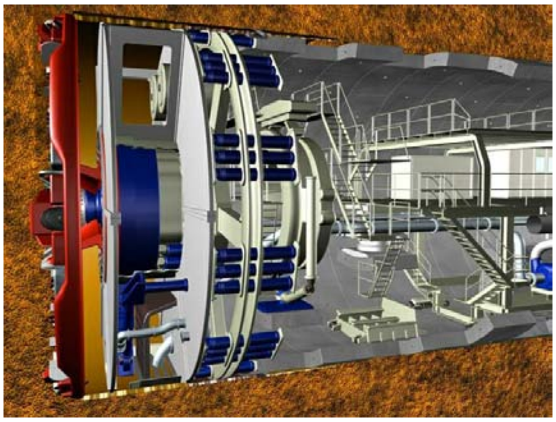

Figure 2. Compound slurry shield machine

This project has some big chalenges, such as the large amount of natural water, big porosity, high compressibility and low intensity in the strata to be encountered. In addition, due to the huge additional, uneven distributed outer load, the strata may deform easily in such geological features as high sensibility, easy thixotropy rheology, thus creats the settlement of stratum and causes the damage of surrounding structures. What's more, there are part of bedrock to be cut in the bottom of the river, which is unfavorable for the shield tunneling.

The diameter of shield machine is 11.3 m (37 ft) and the cross-sectional area of the tunnel is quite big. For the left line, the continuous driving distance is 2550 m (8,200 ft), while the right line 2,499 m (8,200 ft). Such large cross-sectional area and long driving disance shield tunneling project is the first time in China (Figure 2). The largest buried depth of tunnel is 40.5 m (133 ft) and the smallest one is 7.2 m (24 ft). The pressure of soil varies greatly from place to place, and the highest water pressure is 0.57 MPa (83 psi), which requires strong balance ability of shield during construction.

The Bottom Driving of Shield and Security Measures

According to geologic investigation report, the stratum that the shield crosses the segment of the bottom of river is mainly medium-tensity silty fine sand. tense silty fine sand, the middle part of the bottom is cobble layer and high-efflorescence muddy siltstone with sandstone andcleaving stone, and locally the medium-tense and medium-coarse sand, tense medium-coarse sand and plastic silty clay layer. At the river bottom, Yangtze-river cuts through the Quaternary system brand-new pore space supporting aquifer upper plate and the river water and underground water has close hydraulic relationship. The sand in riverbed is shallow water aquiclude its water-stop sheet is Siurian System layer. According to the drilling investigation, the diameter of the cobble is 20-80 mm (0.8-3 in.). The size of the openings on the cutter head of the shield machine and the 400 mm (16 in.) in diameter mud discharge pipeline can let the cobbles pass through smoothly.

When shield crossing the Yangtz-river, the smallest depth of cover is 11.5 m (38 ft). According to the level of the tunnel bottom, the lower part of the working face will cut into dense and solid formation with cobbles, intensive and medium weathered muddy siltstone with sand and shale, so the lower hard formations and the upper soft formations consit of the uneven working face, which would be a great chalenge during the shield driving.

In order to make shield machine driving through Yangtze-river successfully at one time, and decrease the possibility of maintenances and repairs of the shield machine under the bottom of the river, before the shield machine start to work, it should be aligned very carefully. Then after she shield machine entering the reinforcement zone, which is a special area set up for the maintenances and repairs before the machine goes under the river, the shield machine should be inspected comprehensively, especially the cutter head and cutters, and replace the cutters when necessary.

Problems during the construction. The problems should be taken more attention to during the construction are as follows:

1. When shield machine is working in the silt fine sand layers under the river, it's hard to control the soil stability in front of the cutter head. There are many cases can cause unstable of the working face, for example, choosing the improper driving parameters, the poor quality of slurry and the bad effect of back-filling grouting all can cause the settlement of frontage soil easily.

2. When the shield passes through the layer with uneven softness and rigidity may cause:

- The abrasion to cutters is relatively serious
- It will cause the shield machine vibrate up and down.
- The silty fine sand formation on the top of tunnel can collapse easily, which would cause the deformation and settlement of the upper geologic masses.
- It is easy to cause the deflection of the driving direction during the driving process.

3. When the shield passes through the small cover depth section of the bottom of river, it may cause:

- It's easy to cause the leakage of slurry at the end of shield.
- It may cause the discharge of slurry easily at the bottom of river if the pressure of slurry and water to be controlled improperly.
- It may cause the infiltration of water and sand to the inside of the tunnel easily because of the connection between the underground water and river water.

4. When shield machine makes long-distance driving at the bottom of river, it may cause the serious abrasion of cutters and the risk of river water passing from the end of shield into the tunnel at the high hydrostatic pressure sections.

In order to make the smooth passing through of the shield machine**, a**fter making exact detection to the geological data, the following measures are planed to take:

1. The project uses the compound slurry shield machine. When the shield machine is driving, adjust the pressure of slurry to make sure the pressure balanced with the soil pressure and the water pressure of the excavation surface and its stability according to the geology, water condition and the formation pressure. Besides, during the driving process, keep the balance of the amount of excavated mud and the discharged slurry, and enhance the monitor of the amount of the excavated soil in order to prevent the excavation surface from destabilization because of the larger loss of the slurry pressure.

2. When the shield passing through the unstable geologic sections, slower the driving speed and lower the pushing force of the shield and the rotational speed of the cutter head in order to prevent working face from collapse. Generally, the driving speed should be in the range of 20mm/min.

3. Based on the analysis of the geologic data, check the seal of shield tail and the cutters carefully before driving into the unfavorable formations.

4. When driving, slurry and water management is very important, control the largest particle diameter and the particle distribution, and the density and pressure of slurry carefully, and measure the following parameters in time and keep them in control:

- Slurry specific weight;
- Viscosity;
- Yield value(YV)
- Content of Sand in the slurry;
- Filtration characteristics.

5. To improve the quality management of the back-filling grouting, it will be not only necessary to fill the over cut annular space and control the ground settlement, but also necessary to prevent the slurry from being eroded by mud water before having a certain early strength as soon as possible, and control the tendency of the tunnel to move up and shorten the time required for the tunnel to get. When necessary, supplement again with the second grouting, thus making the even grouting to form stable waterproof layer in order to reinforce the tunnel lining.

6. To monitor the collapse of the working face and manage the grouting procedure through the analysis of the soil pressure meter set up at the periphery of the cutter head and the amount of slurry discharged. At the same time through the injected hole at the upper half of the shield machine make grouting to the excavation surface in the range of 30 m (98 ft) from the shield machine and prevent the excavation face of the upper half of the shield from collapse.

7. Operate the shield machine carefully, keep the proper attitude control of the shield machine, control deflection of the shield driving, and prevent the slurry leakage at the end of shield and control the tunnel settlement as required.

8. Enhance the quality management of grouting at the end of shield to prevent slurry from flowing into the tunnel through at the end of shield to cause the collapse and settlement of working face.

In order to make sure the shield passing through the shallow earth covering section of Yangtze-river safely, on the basis of the actual situation of the project, a test driving section was set up under the river bottom to collect and optimize all kinds of construction parameters, make such countermeasures to prevent the tunnel from leakage, settlement, clogging, and deflection of the tunnel , enhance the monitor of the landform settlement at the bottom sector in order to make sure the shield passing through the shallow earth covering area of Yangtze-river safely and the smooth completion of the project.

Preventing slurry leakage. The following measures were used to avoid slurry leakage at the end of shield:

1. Improve the synchronizing grouting quality. Test the grouts and control its initial setting time and its consistency strictly before driving. Control the grouting pressure properly to match the amount of grout and grouting speed with the shield driving speed during the grouting process.

2. Keep the water pressure at the working surface steady and make it a little lower than the grouting pressure in order to prevent the grouts from losing forward.

3. In order to prevent slurry leakage at the end of shield, stick the watertight sponge on the outer board of the concrete segments. So the sponge can swell and seal the leakage when the end of shield leaks slurry.

4. Grout leaps uniformly and sufficiently at the tail of shield. Both automatic and manual grouting methods can be used. The amount of the leap grouted should be a little more than the theoretical need. Based on the calculation, the amount of leap to be grouted should be 120-200 kg (264-440 lbs) per segment drive. If the end of shield leaks, grout leaps manually at the location of leakage immediately.

Preventing discharge of slurry from river bottom. The following measures were used to avoid the discharge of slurry from river bottom:

1. Control the fluctuation range of the water pressure at the working face carefully, which should be controlled in the range of - 0.2 to + 0.62 kg/cm^2 (-3 to + 8 psi).

2. Control the amount of excavation speed efficiently. In principle, the amount of discharged soil should be equal to the theoretical amount of excavated soil. But if control the amount of discharged soil a little less than the excavated, the soil mass in the chamber of the shield can be kept dense, which would be helpful to avoid river water permeating the soil body then to shield.

3. Make strict control on synchronizing grouting pressure and set up the relief valve in the grouting pipeline to avoid breaking the earth covering because of excessive grouting pressure.

4. If the machine breakdowns or can not continue to drive caused by other reasons, the following measures should be taken to prevent the movement of shield machine.

The measures should be taken when the slurry discharged from the river bottom are as follows:

- When the discharge of slurry from the river bottom is detected, if it is not serious, the driving can be continued without lowering the balance pressure. At the same time the advance and segments lining speed can be increased slightly, in order to make the shield machine pass through the slurry-discharging area as soon as possible.
- When the slurry discharging is too serious to continue the driving, the pressure to balance the working face (including the soil pressure and water pressure) should be reduced immediately. Then improve the specific weight and viscosity of slurry. In order to continue the driving process, readjust the relationship between the excavated soil and discharged mud and make them reach a new balance. After a short

distance driving, grout the annular space and readjust the slurry pressure to the normal value and advance forward normally
- When finding the river water flows from the end of the shield into the tunnel, grout polyamine resin correspondingly in the leakage location; at the same time a drainage system should be set up to guarantee the river water came into the tunnel to be drained smoothly and quickly.

Preventing settlement at the river bottom. The following methods can be used to avoid the settlement at the river bottom.

1. Set the slurry pressure at the working face according to the design value and adjust it according to the variation in water level of Yangtze-river during driving. Control the fluctuation of mud water pressure carefully and make sure the slurry pressure is not too lower to balance excavated surface, which will cause the collapse of the working face and then the settlement of soil body.

2. Observe and measure the settlement of the river bottom carefully. When the settlement is larger than 5 cm (2 in.), the synchronizing amount of grouting should be increased properly. In addition, make compensation grouting when necessary.

3. Pay more attention to the quality control of the slurry and to the monitoring of discharged mud when driving under the river bottom. Improve the specific weight of slurry properly and control its viscosity to guarantee the quality of the mud membrane; reinforce the monitor of discharged mud and calculate the relationship between the driving speed and discharged mud in advance. Adjust the working parameters immediately to prevent the settlement from taking place caused by the over cut or abnormally discharged mud.

4. Predict the stability of the front soil body by using the soil body detecting device. When the amount of excavated dry sand is larger than that to be discharged during the driving process, use the stratum detecting device more frequently to analyze and control the settlement of soil body at the working face and adjust parameter immediately according to the specific construction situation, to make the amount of dry sand to be cut close to the theoretical value and reduce the possibility of the collapse of the front soil body.

The Control of the Shield Driving

The quality control of slurry. The proportion of different contents of slurry must satisfy related standards, regulations and the stipulation of technical regulations. For the dense fine sand layer in the project, the bentonite slurry. The proportion (weight proportion) of slurry is bentonite: CMC: calcined soda: water=100:0.28:3.3:700. The function of CMC is to increase the slurry viscosity; and the calcined soda to control the stability of slurry. According to the experience of the river crossing tunnel project in Chongqing, the slurry parameters chosed are as follows: slurry density 1.2 g/cm^3 (75 lbs/cubic foot) or so; slurry viscosity 25-30 s (funnel viscosity); water filtration lost ratio<5%; particle diameter c < 74 μm (2913 microin.).

The control of slurry pressure. The key to prevent the stratum settlement is to keep the pressure balance between the mud water chamber pressure and working face pressure (the sum of earth pressure and water pressure). The adjustment of slurry pressure P is realized through maintaining the balance between the amount of the exacavated soil and that of the discharged mud. It can be achieved by setting up the driving speed and adjusting the amount of discharged mud or setting up the amount of discharged mud and adjusting the driving speed. Slurry pressure P can counter balance with the stratum earth pressure and static water pressure. Suppose the sum of the stratum static water pressure and earth pressure at the center of the cutter head is P_0, generally $P=P_0+20$ kPa (3 psi). Make feedback and adjust to optimize according to geology, embedded depth actual situation and the monitor information of the stratum settlement during the shield driving process.

The control of the shield advancing speed. Choose the proper driving speed to prevent the stress of shield cutter head losing balance because of the faster shield advancing, which would cause the downword moving tendency of the shield and the sand flowing into working face. Generally, the driving speed should be controlled in the range of 5-12 mm/min (0.2-0.5 in./min.).

The Measure of Shield Passing through Yangtze-river One-time

In order to guarantee the shield working at the river bottom in good condition, the overall check, maintenance and repair should be done before it starts to work at the river bottom. There are two reinforcement zone are set up for this purpose. The size of every shield tunnel reinforcement zone is 14.6 m (48 ft) (length) x 17 m (56 ft) (width) x 17 m (56 ft) (height), while the size of the contact passage reinforcement zone is 11.4 m (37 ft) (length) x 5.99 m (20 ft) (width) x 13.2 m (43 ft) (height), both adopt three-tube spin spray pile and compressed grouting to reinforce. The reinforced height is 3.5 m (16 ft) on shield tunnel, and 2.5 m (8 ft) under the tunnel, while the total height is 17 m, and the length of each pile is 35.9 m (118 ft).

Two rows of compressed grouting holes are arranged densely at the out board of spin spray pile to press the grouting densely for reinforcement and waterproofing. In order to avoid the reinforcement dead zone because of the deviation of the site of spin spray pile inside the reinforcement zone, calculate the bite situation of the reinforcement zone according to the derivation measurement after the drilling, while set up the compressed grouting hole at the part with worse bite situation to press and reinforce. It sets up 138 grouting poles, and the reinforcement the length of the hole is 16 m (53 ft). There are 118 three-tube high pressure spin spray piles in the contact passage reinforcement zone, and the height of the reinforcement zone bottom equals with that of the shield tunnel, while the total height is 132 m (433 ft), and the hole length of each pile is 39.7 m (130 ft). Besides, there are 18 grouting holes with 40.2 m (132 ft) in length each, the reinforcement section is 12.2 m (40 ft). The construction method of the reinforcement zone is as the third section of chapter three. After finishing reinforcement check the reinforcement effect and intensity by drilling, and check the water percolating capacity by pumping test. Supplement the compressed grouting when necessary.

Conclusions

The surrounding conditions of Wuhan Yangtze-river tunnel are very complicated, and the project and the applied techniques are difficult. The main purpose of this paper was to discuss the potential problems and present the corresponding construction measures according to the construction experience of authors and related references.

Bibliography

Lu, Z.Q., and Wang, C. F. Key Construction Technology of Shield TBM Launching Shaft of Wuhan Yangtze River Crossing Tunnel. *Tunnel Construction* .2006 (1)

Zhang, J. Construction Technology of Chong Qin Yangtze River Crossing Tunnel. *West-China Exploration Engineering*. 2006 (2)

Huang, W.N. Collapse Treatment under River in a Project of Slurry Shield Tunneling. *Modern Tunneling Technology*. 2006 (2)